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THE PROCESS CONTROLLER's GUIDE TO

WASTEWATER TREATMENT

This is number 3 in the Process Controller Guide series of documents

Number 1	Pollution Control.
Number 2	Water Sources and Water Treatment.
Number 3	**Wastewater Treatment**

These notes were prepared for delivery to persons studying for the Diploma in Water Care Technology offered by the then Cape Technicon, in the early 1980's.

Since the time of writing, new Acts of Parliament, Guidelines and Regulations have been implemented. These have NOT been incorporated into this document. Persons interested in the latest Acts, Guidelines and Regulations need to look elsewhere for these.

It is intended that this document be a useful reference and training manual guide to all persons involved in the Water and Wastewater Industry.

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These documents are dedicated to the thousands of men and women (both present and past) who are involved in the life critical profession of Water and Wastewater Treatment.

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Having been written some 38 years ago; new information, techniques, analytical methods have since become available. In spite of this, it is felt that this document will to continue to provide the reader with a firm basis from which they will understand and appreciate the water cycle and related matters.

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THE PROCESS CONTROLLER's GUIDE TO

WASTEWATER TREATMENT

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WASTEWATER TREATMENT

PART 1.

SOURCES AND COMPOSITION OF WASTEWATER.

1.1 DOMESTIC WASTEWATER.

This is the wastewater from the kitchen, bathroom, toilet and laundry. To the mineral and organic matter already in the water supplied to the community, is added a load of human wastes, paper, soap, dirt, food wastes, sometimes including garbage from garbage grinders, and other substances. Some of the waste matters remain in suspension, some go into solution and others are, or become, so finely divided that they acquire the properties of colloidal particles.

1.2 INDUSTRIAL WASTEWATER.

These vary very widely in quality and quantity depending on the industrial operation from which they were derived. They vary from relatively clean rinse waters to others that are so objectionable or toxic that they should not be admitted to the public sewerage system. Some industrial effluents, such as cooling water, may be of such high quality that they may be re-used on the premises for irrigation.

1.3 INFILTRATION WATER.

This is water that enters a separate system of sanitary sewers unintentionally. This arises from 3 main sources:

1.3.1 Direct infiltration.

This is where down pipes from roofs are connected into the sanitary sewer system instead of the stormwater system. This type of infiltration occurs only when it is actually raining and for a short period after it has stopped. The water entering the sewer is usually of high quality.

1.3.2 Surface Water Infiltration.

This arises from flooding of areas when the water enters the sewers via a manhole cover and also where stormwater drains across an area and then enters the sewer. This type of infiltration will start sometime after rain has stopped. This water can contain sand, grit, soil etc.

1.3.3 Groundwater Infiltration.

This arises from imperfect joints, between pipe and manhole and between pipes as well as in the wall of the manhole. This infiltration is dependent on the relative levels of the sewer and the water table. Infiltration will take place only when the water table is as the same level as the sewer. The water table usually rises during the wet season, but the height of the water table may be dependent on the movement of water into or from the area, where the sewers are laid.

Groundwater infiltration may start sometime after the rains have stopped. The quality of the groundwater that infiltrates into a sewer may vary through very wide limits. In sewers laid near the sea, groundwater with salinities as high as that of seawater, may enter the sewer. Only small quantities of seawater need enter a sewer to produce a measurable increase in dissolved solids content of the wastewater.

It is necessary to differentiate between the various types of infiltration because of their different effects on the flow of wastewater in the sewer. The increase in flow in the sewer is most sudden as a result of direct infiltration, and usually more gradual with surface water infiltration except when flooding occurs. The latter effect starts later than the former and continues for longer. Rates of change in groundwater infiltration are usually very gradual, for example groundwater infiltration may start days or even weeks after the rainy season has started and may continue for months after the rains have stopped.

Besides increasing the volume of the wastewater to be treated, infiltration water can have a significant effect on the chemical composition of the wastewater. The dissolved solids content can be increased dramatically while the oxygen demand per litre is usually reduced.

1.4. SEWER SYSTEMS.

There are two main types of sewer systems in use:

1.4.1 SEPARATE SYSTEMS.

Here domestic and industrial wastewater is carried in one set of pipes while stormwater from houses, streets, paved areas etc., is carried in another set of pipes. In this system, the municipal wastewater will be discharged at a wastewater treatment works or into the sea or some other suitable site. The stormwater system will discharge into a river, lake or the sea.

Nearly all recently installed sewer systems are of the separate type. Although the first flush of stormwater off roads etc., may be fairly highly polluted with grit, oil and other organic and inorganic wastes, the effect of this is generally much less than allowing a combined sewer system to overflow into a river because the treatment works cannot handle the flow above a certain value.

In a country such as South Africa, where the rainfall is usually fairly intense, the rate of runoff can be very high. In using a separate system, the sanitary or foul sewers may be kept fairly small while the stormwater may be ponded in retention ponds in certain areas in order to reduce the rate of runoff. This has the advantage of reducing the size of the pipes and hence their cost. Such stormwater detention ponds would not be possible if a combined system were used. The use of stormwater detention ponds also allows the recharge of groundwater through infiltration.

1.4.2 COMBINED SYSTEMS.

Here the municipal wastewater and stormwater are carried in the same set of pipes. It is important to build a wastewater treatment works to handle the total flow that may be discharged via a combined sewer system. If a combined system is in use, any flow in excess of a predetermined figure usually about 3 times the average dry weather flow, is discharged into stormwater tanks at the treatment works. It may be necessary to have overflows within the sewer system; these overflows would discharge untreated effluent into a river, lake or sea.

One major disadvantage of the combined system is that during the dry period the sewers run at a very low level and settlement of grit and detritus can take place. When the rain occurs and the flow rate increases, this previously settled material is re-suspended and can create a suddenly large load on the treatment works just at the time when the hydraulic load is increasing. This can result in the works becoming overloaded. If the works is designed large enough to handle these peaks, then these will be excessive spare capacity during dry periods.

Certain items of equipment such as storm tanks, bypass sewers etc. will only be used for a part of the year and will lie dormant for the other part. This can result in the equipment not being operational when required.

Combined systems are usually found only in areas sewered some time ago. The modern trend is to use the separate system.

1.5. COMPOSITION OF WASTEWATER.

1.5.1 Organic Constituents.

Raw wastewater contains a very large variety of organic constituents. These will vary from simple compounds such as ammonia to complex organics such as proteins and steroids in domestic wastewater. In industrial wastewater virtually any organic compound could be found depending on the types of industries in the catchment area.

The concentration of organic materials can vary from less than 10 mg/L up to more than 3 000 mg/L, in the case of ammonia found in the contents of nightsoil buckets or pit latrines.

1.5.2 Inorganic Constituents.

In contrast with organic constituents, the inorganic constituents of the reticulated drinking water are significant, usually in the range of 100 - 500 mg/l measured as the total inorganic dissolved solids.

The increase in inorganic content of domestic wastewater over the intake water is as the result of ingestion and use of table salt, use of soap and detergents etc. Most detergents contain a significant quantity of inert filler material such as sodium sulphate. Most of the added inorganic matter is sodium chloride and sodium sulphate. In industrial wastewater, potentially very much higher concentrations of inorganic constituents may be discharged. These may have to be prohibited from entering the sewers and be treated on site, or removed to a suitable disposal site.

Nearly all inorganic constituents pass through a wastewater treatment works virtually unaltered, and as the concentration of these salts in the final effluent is limited by law, it is often necessary to restrict the input of these salts. Sodium, particularly, reduces the usefulness of the treated effluent for irrigation purposes. Unfortunately, it is rare that a sodium salt used in industry can be replaced by another cation due to cost and solubility problems. Good housekeeping and elimination of excessive use of chemicals in industrial processes can go a long way to reducing the sodium load of the effluent.

1.5.3 Oxygen Demand.

Most inorganic substances which can be reduced in an oxidation-reduction process are present in a reduced state when entering a treatment works. Not necessarily their most reduced state. This is because when they are manufactured either by nature in the photosynthesis process or by artificial means, energy is fed into the substrate. This means that in combination with oxygen and under the correct conditions, these substances will burn, become oxidised and release energy. Usually however, the substances are too cool and too dilute to actually burn. The dilution is usually being as the result of the presence of water.

Before the effluent can be discharged into the receiving water it is necessary to remove a large proportion of this energy. This is what the wastewater treatment works does. Unfortunately, due to many factors, it is necessary to feed in energy to extract the inherent energy. There is a net energy output, but most of it is in a dilute form and cannot be recovered.

In order to monitor the treatment process, one needs to know the energy present in the feed to the works and at various stages of the treatment process. There is not one test or determination that will give the whole answer.

Reduced organic substances can generally be split into two groups:

1. carbonaceous, that is containing carbon in addition to oxygen and hydrogen;
2. nitrogenous, that is containing nitrogen in addition to oxygen and hydrogen.

In both cases small quantities of sulphur and phosphorus may be present. The quantity of oxygen needed to oxidise all carbon to carbon dioxide, the hydrogen to water and the nitrogen to nitrate is called the oxygen demand. For reasons which will be seen below this is usually split into the carbonaceous oxygen demand (This is NOT abbreviated as the COD - see later) and the nitrogenous oxygen demand sometimes abbreviated to NOD., The sum of these two is known as the total oxygen demand or TOD.

There are tests available to determine the oxygen demand due to carbonaceous material. The tests are, however, not equivalent.

1.5.4 The Chemical Oxygen Demand (COD).

This test uses a strong oxidising agent, acid dichromate, to oxidise all carbonaceous material in the sample in 2 hours. This has the advantage that the result may be known in a total of about 4 hours,

Nearly all carbonaceous organic materials are completely oxidised. Certain inorganic constituents such as the ferrous iron and inorganic sulphides are also oxidised. This test is known in some parts of the world as the Dichromate Value (DV).

The main disadvantage of this test is that it does not differentiate between organic substances which can be decomposed by organisms and those which cannot. The nitrogenous oxygen demand is not detected by this method.

Chlorides interfere with the test but mercuric sulphate is used to complex the chloride up to 2 000 mg/L. Silver Sulphate is used as a catalyst to hasten the oxidation of certain chemicals.

1.5.5 The Biochemical Oxygen Demand or BOD.

This makes use of the bacterial action reducing the dissolved oxygen content, but the period of reaction is fixed at 5 days at 20ºC.

The samples are diluted as needed with special dilution water that is just saturated with respect to dissolved oxygen content and usually contains a seed of suitable active organisms plus some essential minerals such as magnesium and iron. Here the organisms use the dissolved oxygen present to oxidize the substances present in the wastewater. The dilutions are made on the basis of prior knowledge and are made so that between 30% and 70% of the initial dissolved oxygen content remains at the end of the 5 day period. This is why the name is sometimes written as $BOD_{5.}$ It is often advisable to put on more than one dilution, as the test cannot be repeated on the same sample. The initial and final dissolved oxygen contents may be determined with a dissolved oxygen meter or using the Winkler method.

A major disadvantage of the test is the 5 days waiting period for the results. For this reason, this test is not used that often these days.

The BOD test is invalid when certain industrial effluents are present as they can exhibit an inhibitory effect on the micro-organisms. The oxygen demand due to carbonaceous material will be satisfied first and the oxygen demand due to nitrogenous material will be satisfied later. This latter step may or may not start during the normal 5 day incubation. In order to determine the oxygen demand due to carbonaceous material only, nitrification may be stopped by the addition of a small quantity of Allyl-Thio-Urea, ATU.

Sometimes the BOD test is allowed to continue for 30 days at which stage all the biodegradable carbonaceous oxygen demand and the nitrogenous oxygen demand will have been satisfied. This has been called the ultimate oxygen demand or UOD or occasionally BOD_{30}.

1.5.6 Organic Carbon Content.

The above two tests have the disadvantage of being time consuming and expensive to perform. The COD takes about 4 hours and uses expensive chemicals and the BOD takes 5 days.

Using an organic carbon analyser enables one to obtain a result within minutes. Here a small sample is injected into an instrument when all carbon is burnt to carbon dioxide, the latter being detected and quantified.

1.5.7 The Permanganate Value or Oxygen Absorbed.

This analytical method is very rarely used these days. The original name of the test was Oxygen Absorbed. As there were two slightly different tests being used, the test used in South Africa became known as the Permanganate Value or PV_4. The “4” refers to the 4 hour duration of the test. This test uses a less vigorous oxidation process than the Chemical Oxygen Demand. In this test only the more readily oxidizable substances are actually oxidised in the time allowed for the test. As with the COD test, no differentiation in made between biodegradable and non-biodegradable substances.

It is not possible to compare the results obtained by the one method with those obtained in another method, as each method determines different proportions of the oxygen demand. For raw (unsettled) municipal wastewater containing up to about 25 % of industrial wastewater of a wide variety, the following ratios have been found to be appropriate.

$$\text{COD} : \text{BOD} : \text{PV}_4 :: 12 : 6 : 1$$

$$\text{COD} : \text{BOD} :: 2 : 1$$

The nitrogenous oxygen demand cannot be determined directly except possibly by determining the difference between the UOD and the BOD-ATU results. This test takes 30 days.

Usually the nitrogenous oxygen demand is taken as 4.6 times the Kjeldahl nitrogen content. The multiplying factor being obtained from the equation:

$$2NH_4 + 4O_2 \rightarrow 2H_2O + 2NO_3^- + 4H^+ + 2e.$$

The total oxygen demand is usually taken as the sum of the chemical oxygen demand and the nitrogenous oxygen demand calculated in terms of the Kjeldahl nitrogen content.

WASTEWATER TREATMENT.

PART 2

GENERAL INFORMATION.

2.1 PER CAPITA DISCHARGE VOLUME.

The per capita discharge of domestic wastewater varies widely. The smallest volume being from the nightsoil bucket or stercus pail of the type used in emergency or temporary camps. Here the volume of liquid waste attributable to one person is 1 - 2 litres per day. The per capita discharge from houses that are connected to sewerage systems varies from about 70 L/head-d in sub-economic units, up to more than 300L/head-d in expensive blocks of flats where water is not separately metered. The averages for the various incomes groups are given in Table 1.

TABLE 1 – TYPICAL WASTEWATER VOLUMES BY INCOME GROUP.

INCOME GROUP	DAILY WASTEWATER PRODUCTION
Lower	90 L/head-d
Middle	150 L/head-d
Upper	200 L/head-d

The taking of baths regularly, the use of washing machines and dishwashers increases greatly the quantity of domestic wastewater.

The above figures relate to domestic discharge of wastewater, to the above must be added a contribution of domestic wastewater from industrial and commercial areas. This is usually taken as 50 litres per working person per day. No allowance has been made for industrial effluent. This must be borne in mind when abstracting so called per capita wastewater discharge figures from the literature. Some authorities take the total wastewater discharged per day and divide by the resident population in the town or area. On this basis, the so-called per capita flows of over 400 litres per head per day, are possible.

2.2 PER CAPITA SOLIDS LOADING AND OXYGEN DEMAND.

The organic load per capita does not vary over such wide limits. The per capita pollutional load is usually taken as 110 g Chemical Oxygen Demand per capita daily or 55 g Biochemical Oxygen Demand per capita daily. The range of values found is probably +- 20 %. A typical breakdown of this pollutional load is given in Table 2.

TABLE 2 – TYPICAL BREAKDOWN OF POLLUTIONAL LOAD IN DOMESTIC WASTEWATER.

Settleable Solids	19 g/pe-d
Non-Settleable Solids	23 g/pe-d
Total Suspended Solids	42 g/pe-d
Dissolved Solids	13 g/pe-d
Total Solids	55 g/pe-d

There is some disagreement as to these values; the per capita water usage does cause these values to vary. With higher water consumption the non-settleable solids and dissolved solids tend to increase at the expense of the settleable solids.

As may be expected, the domestic wastewater flow is not constant throughout the day and night. There are peaks at the time when most people get up and a smaller peak when most people go to bed. Minimum flow is usually about 04h00. A typical pattern for flow and COD value is shown in figure 1 - for dry weather flow. The pattern may vary widely depending on such factors as a large residential area being some distance from the treatment works so that the flow peak from that area is delayed some hours

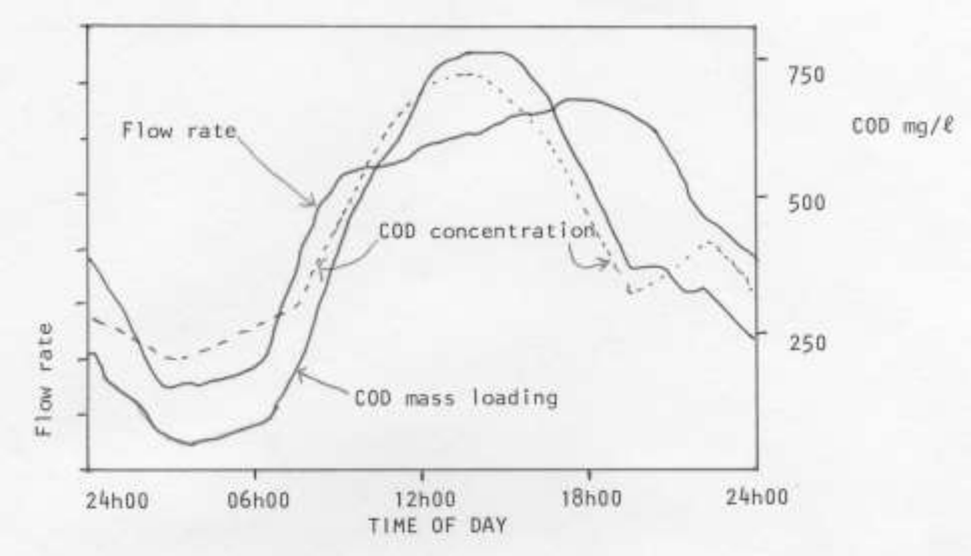


Figure 1: DIURNAL VARIATION IN FLOW AND CHEMICAL OXYGEN DEMAND.

2.3 THE HISTORY OF WASTEWATER TREATMENT.

The historical sequence of engineering constructions for the biological treatment of wastewater proceeded from sewage farms through intermittent sand filters and contact beds to biological filters or trickling filters to activated sludge. It is understandable that treatment began as a farming practice. The wastewater that poured onto fields from urban drainage systems could irrigate and enrich soils that were thirsty for water and hungry for nutrients. When land became scarce and financial returns on wastewater irrigated properties became unattractive then this method of disposal lost favour.

On natural sand areas, such as the Cape Flats, the intermittent sand filtration method of wastewater treatment may be continued for extended periods. If the wastewater was treated by settling before being applied to the ground it has been found that the dosage rate could be increased - this is understandable.

On heavy clay soils, this method of disposal ran into problems very quickly and it was necessary that the wastewater be treated before being discharged onto these soils. This is where the contact bed started to be used. These were shallow tanks filled with slag or stone. These beds within tanks were well suited for the colonization of large populations of organisms which, in their foraging, removed from the wastewater the non-settling and dissolved organic compounds.

Later it was found that the effective loading and performance of the beds, which were operated on a fill and draw and a resting cycle, could be raised appreciably by the addition of sprays that discharged the incoming wastewater onto the contact media in substantially constant streams with appreciable dissolved oxygen content. No longer were the beds filled with wastewater, instead they were left open to the air throughout their depth so that the wastewater might trickle over the contact surfaces to the underdrainage system while air swept through the beds and kept them aerobic. These so-called bacteria beds then became known as biological filters or bio-filters for short. Other names used are percolating filters or trickling filters.

In the 1920’s, the activated sludge process was developed in which the wastewater and flocs of micro-organisms were supplied with air. This wasfor the dual purpose of keeping the units aerobic, in spite of the heavy concentrations of living organisms and their high oxygen demand, and the flocs in suspension.

Over time the activated sludge process was improved with nitrification (oxidation of ammonia to nitrate) and de-nitrification (reduction of nitrate to nitrogen gas). Later still, biological excess phosphate removal was developed, where under suitable conditions, certain organisms can remove virtually all soluble ortho-phosphate present in the wastewater.

Where communities had enough money to pay for the construction and operation of a wastewater treatment works, two main types were available viz biological filtration and activated sludge. Small communities, situated principally in rural areas, were seldom able to afford a mechanical treatment works. A suitable system in this case, was the septic tank with sub-surface irrigation for installation in individual properties. However these systems were not suited to the diffusion of wastewater into clay soils or areas where the soil cover of the underlying rock was thin.

With the increased availability of piped water and the consequent greater water use, the absorptive capacity of even moderately permeable soils became over-taxed. Better treatment systems were needed for wastewater disposal for small rural communities. This was provided by the development of the oxidation pond, also called a stabilisation pond.

More recently small rotating disc units and activated sludge works have become available, often made of steel units, to treat population equivalents down to as few as 50 persons.

2.4 EFFLUENT STANDARDS.

The standards to which the effluent from a wastewater treatment works shall comply are detailed in the permit issued by the Department of Water Affairs to the owner of the treatment works.

As a wastewater treatment works is classified as an industry, the Regional Standards for Industrial Effluents - Government Notice R553 of 5 April 1962 applies. In this there is a Special Standard applicable when effluents are discharged into a Scheduled River, and a General Standard applicable elsewhere.

2020 note: new standards and Water Licence Requirements now apply.

The requirement of the 1956 Special and General Standards are detailed below.

TABLE 2 – GENERAL AND SPECIAL STANDARD IN TERMS OF THE WATER ACT (Act 54 of 1956).

	Units	Special Standard	General Standard
pH value		5.5 - 7.5	5.5 - 9.5
Dissolved Oxygen (% of saturation)		>75	>75
Typical (Faecal) Coli	MPN/100mL	Nil	Nil
Temperature	°C	25	35
Chemical Oxygen Demand	mg/L	30	75
Total Dissolved Solids above intake	mg/L	NOTE 1	500
Suspended Solids	mg/L	10	25
Sodium as Na	mg/L	50	50
Soap, Oil & Grease	mg/L	Nil	2.5
Residual Chlorine as Cl	mg/L	Nil	n.s.
Free & Saline Ammonia as N	mg/L	1.0	10.0
Nitrates as N	mg/L	1.5	n.s.
Arsenic as As	mg/L	0.1	0.5
Boron as B	mg/L	0.5	1.0
Total Chromium as Cr	mg/L	0.05	0.5
Copper as Cu	mg/L	0.02	1.0
Phenolic compounds as C_6H_5OH	mg/L	0.01	0.1
Lead as Pb	mg/L	0.1	1.0
Iron as Fe	mg/L	0.3	n.s.
Manganese as Mn	mg/L	0.1	n.s.
Cyanides as CN	mg/L	0.5	0.5
Sulphides as S	mg/L	0.05	1.0
Fluoride as F	mg/L	1.0	1.0
Zinc as Zn	mg/L	0.3	5.0

NOTES

1. Not to be increased by more than15% above that of the intake water;

n.s. = not specified

In addition there is a special phosphorus standard that is applicable in certain areas. Here the limit is 1 mg/L of ortho-phosphate.

WASTEWATER TREATMENT.

PART 3.

PRELIMINARY TREATMENT

3.1 INTRODUCTION.

Wastewater contains polluting matter in many forms. In addition to dissolved impurities, it contains material in suspension and in colloidal form which can be broadly divided into organic and inorganic material. The size of particles of suspended matter also varies widely from 1 - 2 mm up to over 20 mm.

Large solids and grit can cause blockages, damage and wear to pipes, valves, pumps and treatment equipment. The aim of preliminary treatment is therefore to protect the principal treatment processes which follow. It should be borne in mind that the essence of such treatment is protective, and no more material than is necessary, should be removed at this stage.

Although preliminary treatment will be discussed with particular reference to wastewater treatment, many of the types of equipment mentioned are suitable for, and widely used in, the pre-treatment of industrial wastes at the factory prior to their discharge to the foul sewer or, in a few cases, directly to a watercourse.

The presence of industrial wastes and the efficiency of their pre-treatment at the factory will influence the nature of the polluting matter.

There is a diversity of practice as the sequence of preliminary treatment processes, i.e. whether grit removal shall be in advance of screening or vice versa. If grit removal is placed first, the equipment used suffers from accumulations of rags and paper which festoon the inlet vanes or the paddles of the grit removal unit and tend to collect in the valves, pipelines and pumps of other grit-removal devices. In general, the present-day practice is to install screens prior to grit-removal tanks.

In designing preliminary treatment processes, provision should be made for measuring and recording both the rate of flow and the running total of the wastewater arriving at the treatment works. Facilities should be provided to enable representative samples to be obtained at all stages of treatment.

3.2 SCREENING AND REMOVAL OF SOLIDS.

To minimise difficulties caused by large solids, they may be:

1. removed entirely by screening;

2. removed and returned to the flow after their size has been reduced by screening and maceration;

3. reduced in size by maceration without removal from the wastewater.

3.2.1 Screening only.

Bar screens or racks are usually constructed as a series of metal bars arranged in one plane across a slightly expanded channel and are often inclined at an angle in a downstream direction. See Figure 2.

S.A.M.E.

Figure 2 – A TYPICAL HAND-RAKED BAR SCREEN.

Bar screens fall into two groups:

1. coarse - having spacings wider than 38 mm;
2. fine - having spacings 10 to 25 mm wide;
3. very fine - having spacings less than 6 mm wide.

The bars are either circular or rectangular in section. Occasionally they are tapered rectangular, to prevent jamming of solids between the bars.

On the smaller works i.e. about 1 000 m^3/d or less, it is usual to have only a hand raked screen. On the larger works it is usual to have one or more mechanically raked screen with one hand raked screen for use during excessive flows or during power failures. The hand raked screen is usually a coarse screen. The angle of inclination of hand raked screens is usually 30° to 45° to the horizontal. This increases the effective area of the screen and facilitates the removal of screenings.

Mechanically raked screens may have straight or curved bars depending on the action of the rake. Many different patterns have been developed as each manufacturer has tried to eliminate the problems of its predecessors. These days the tendency is to reduce or even to eliminate moving the parts under water - sooner or later these give trouble.

The depth of the floor below ground level will influence the type of mechanical screen being used. The curved bar screen is usually limited to cases where the floor is not more than 2.4 m below ground level. Straight bar screens of the vertically or near vertically raked type have been used where the floor is nearly 6 m below ground level.

As screenings accumulate the difference in the level of the wastewater upstream and downstream of the screen will increase, this can be used automatically start the action of the rake, when the head loss reaches a predetermined limit usually 75 to 100 mm. The rake may also be initiated by a pre-set timer. It is important to prevent surcharging of the sewer upstream of the screen to a point where overflows in the lower parts of the sewer system start to occur. A typical mechanical screen is shown in figure 3.

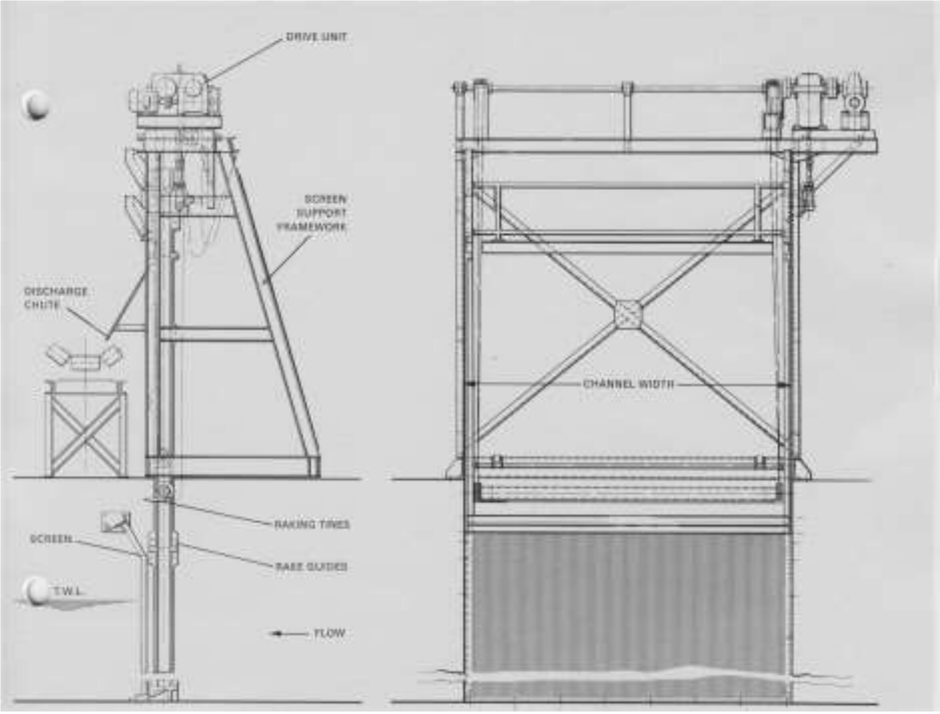
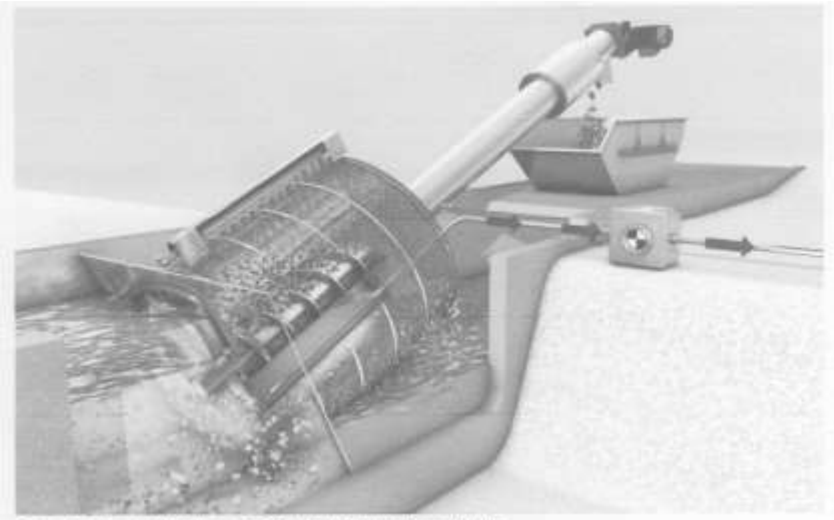


Figure 3 – A TYPICAL MECHANICAL SCREEN.

The modern trend is to screen 6mm size, as this removes organic material as well, the screened solids and washed and often compacted to squeeze out excess water and are sometimes bagged before being placed in a container for later removal to a landfill site.

Rotating screens are sometimes used. There are two main types:

1. Cup screens - where the wastewater passes through the screen from the inside;
2. Drum-screens - where the wastewater passes through the screen from the outside. A typical example is shown in figure 4. Drum screens usually have a 3 mm slot width.



Schematic diagram of the ultra-fine ROTAMAT™ Membrane Screen RoMem

(HUBER Rotamat)

Figure 4 – A TYPICAL DRUM SCREEN.

3.2.2 Screening and Maceration.

The process of removing solids from the flow, mechanically reducing their size and returning them to the flow used to be quite popular, but is quite rarely used these days. The solids removed contained grit that resulted in significant wear on the disintegration equipment while plastics were cut up and passed through the screen and accumulated later in the treatment process or even passed out with the final effluent. When screening and maceration were used together, the usual practice was to remove the screenings from the wastewater using a mechanically raked screen and to pass the screenings into a macerator to be reduced in size. It is necessary to draw water from downstream of the screen as a carrier, the cut-up screenings and the water are then returned upstream of the screen so that if some material has not been chopped up small enough then hopefully it will be screened out again and pass through the macerator a second time. Some of the water discharged from the macerator is used to wash the screenings from the screen into the macerator.

3.2.3 Maceration only.

This process is very rarely used these days as the equipment must be sized to handle peak wet weather flows. This made it unnecessarily expensive. Again plastic materials were cut into smaller pieces that accumulated elsewhere in the system.

3.3 SCREENING QUANTITIES.

The quantity of screenings present in domestic wastewater varies over a wide range. Peak daily collection may vary from 2 to 5 times the average. As may be expected, the finer the screens the larger the quantity of material removed. Certain industrial effluents can increase these figures significantly. Typical quantities for domestic wastewater are given in Table 3 below:

TABLE 3 – TYPICAL SCREENINGS PRODUCTION WITH DIFFERENT SCREEN SPACINGS.

Clear Opening mm	1 - 6	12 - 25	25 - 50
litres screenings / ML flow	60 - 240	20 - 40	5 - 20

The channel in which screens are installed should ideally be narrow enough to keep the minimum velocity of approach of flow above about 0.3 m/s to prevent the deposition of grit, and wide enough to ensure that the maximum velocity of approach does not exceed about 1 m/s, to prevent screenings which have already been intercepted from being dislodged and forced through the screen. Under normal flow the velocity of approach should be about 0.5 m/s.

Where wastewater is discharged untreated or partially treated, e.g. by settling only, into the sea through a sea outfall, fine screens are sometimes installed on the outfalls. These are used to remove unsightly floating matter and for removing solids that decrease the efficiency of disinfection using chlorine. This will be covered later. Fine screens are also recommended for use in certain industries prior to their discharging effluents into the sewers.

The use of fine screens in place of primary settling tanks for treating domestic wastewater is not popular in South Africa because they are inefficient, expensive to maintain and the disposal of the screenings causes difficulties.

3.4 HANDLING OF SCREENINGS AFTER REMOVAL.

The modern trend is to wash the screenings after removal from the flow. This returns organics such as faecal solids, for treatment and thus reduces the odour producing and fly attracting potential of the screenings. In many cases, the washed screenings are pressed to remove free water. This further reduces the odour producing potential. The wash water may be treated effluent or fine screened or filtered raw wastewater. The latter will generally only be used as sea outfalls that have only preliminary treatment.

3.5 GRIT REMOVAL – GENERAL.

Grit may be defined as the heavy mineral matter in wastewater, such as silt, sand, gravel, cinders, metal and glass. It is abrasive in character and causes erosion of pumps and other equipment if not removed. It can also cause blockages in sludge lines and will accumulate in digesters reducing the active volume. This would cause a reduction in the available volume in the digester and would reduce the actual sludge retention time.

The composition of the grit may vary seasonably, at certain times of the year the wastewater may contain fruit stones and vegetable debris, eg. watermelon pips and soil originating from vegetable washing and preparation.

As the density of grit is much greater than that of the wastewater by which it is transported, there will always be the tendency for it to settle out when the rate of flow is low, e.g. during the night. There is also a tendency for grit to settle out in pump station sumps but this can be minimized by careful design. In sandy areas, such as the Western Cape, the quantity of grit can be surprisingly high. Grit loads of up to 0.2 m^3/ML may be found. A large amount of sand enters at gulleys and catch pits outside bathrooms and kitchens. In some areas, where sewers are being laid and houses being connected up, sand enters during the actual connection and often where stubs are not properly capped while awaiting a future connection. Cracks in manholes or pipes may allow ground water containing sandy material to enter if the water level is above that of the sewer.

3.6 METHODS OF GRIT REMOVAL.

3.6.1 The Old Fashioned Method - The Detritus Tank.

On many of the older works, mainly overseas, detritus tanks were used. These were "designed" and installed before the theoretical concept of grit settlement was established. These detritus tanks are basically settling tanks with somewhat arbitrary dimensions and a capacity of 1 % of the average dry weather flow.

Usually they removed most of the grit and a large proportion of the heavier organic matter so that an evil-smelling "detritus" had to be dealt with. The deposited material had to be removed either manually, by bucket elevator or by a mechanical grab fitted to a travelling gantry. Their design did not permit the efficient removal of grit from the wastewater over a wide range of flows. At low flows too much organic material tended to settle out while at high flows there was grit carry over into the next stage of treatment.

This detritus created disposal problems because of its smell and fly breeding. Incineration is the best method of disposal. These tanks are no longer installed and in many cases they are being replaced by a more modern method of grit removal.

3.6.2 The Modern Methods.

There are 3 main types of grit removal systems in use today:

1. the horizontal flow grit channel, with or without aeration - here the wastewater enters at one end and passes along an essentially straight line to the discharge end, and is then discharged;

2. the square tank - here the wastewater enters across one side of the tank and is discharged at the other side of the tank, again the flow path is essentially a straight line;

3. the circular tank - here the wastewater enters at a point and is made to rotate in a horizontal plane and is then discharged at another point or over a weir.

The problem of separating grit from the wastewater and leaving the lighter organic material for subsequent removal is one of flow control. The optimum velocity of the wastewater for the settlement of grit has been found to be 0.3 m/s, and the problem is how to achieve this velocity at all rates of flow. A complicating factor is that fat and oil adsorbed onto the grit lowers its density and thus reduces its settling rate.

3.6.3 The horizontal flow grit channel without aeration.

1. Velocity control by means of a vertically throated flume.

In this system the flume is located immediately downstream of the grit channel to control the depth of flow in the channel. If the shape of the channel approximates a parabolic cross-section, so that the cross-sectional area of the channel at any level is proportional to the rate of flow then the velocity will be essentially constant for all flow rates. To construct a grit channel to an exact parabola is almost impossible, it is usual to approximate the shape by using five tangents - as shown in Figure 5. An actual layout is shown as Figure 6

Figure 5 – A TYPICAL GRIT CHANNEL PROFILE.

(*DWAF*)

Figure 6 – A TYPICAL HOROZONTAL GRIT CHANNEL PAYOUT.

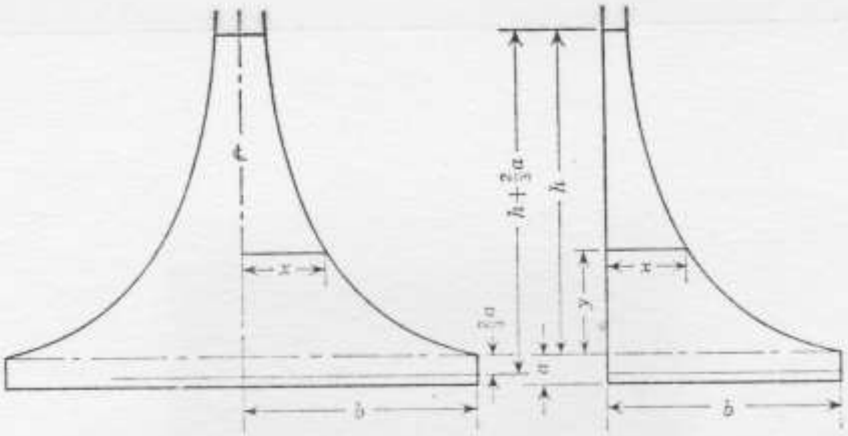
It is usual to construct a works with at least two grit channels. The grit channels may have individual flumes, or all may discharge through a common flume. At many works a flow meter is installed at the common flume.

At works where the maximum wastewater flow rate is very much greater than the minimum flow rate, such as those works where a combined system of sewers, it may be necessary to use one grit channel at low flow, two channels at higher flow and even three channels at storm flow.

The operation of additional grit channels may be manual or preferably be controlled by electrically operated penstocks actuated by a flow recorder on the control flume or flumes. The flow recorder is set up to give the necessary signals at the predetermined rates of flow.

2. Velocity control by means of a proportional flow weir.

This type of horizontal flow grit channel is square or rectangular in cross section. The proportional flow weir is installed vertically at the discharge end of each grit channel. See Figure 6.



(a) PROPORTIONAL WEIR (b) SUTRO WEIR

—ELEMENTS OF PROPORTIONAL AND SUTRO WEIRS

Figure 7 – PROPORTIONAL FLOW CONTROL WEIR.

As above, more than one channel may be required. If a flow meter is required then a separate measuring flume is needed. This flow meter may then be programmed to switch in additional grit channels as needed.

The removal of grit is usually carried out manually on smaller works. On the larger works, grit removal is by pumps, vacuum suction or by dredger.

3.6.4 The horizontal flow grit channel with aeration.

In this channel, the wastewater travelling along the chamber is made to progress in a spiral motion by applying compressed air along one side of the chamber. The design of the chamber is such that a velocity of 0.3 m/s is achieved near the bottom of the chamber, thus allowing grit to fall into hoppers beneath the main floor. The settled grit is removed by air lift or other pumps which discharge to a storage compartment from where it may be removed as required.

The capacity of the aerated grit chamber usually provides for two minutes detention time at maximum wastewater flow, and the velocity of 0.3 m/s. It is usual to use 1 - 2 m^3/h of air per m^3 of total grit chamber capacity.

If the air was not used then at low flows much organic material would settle out, and this would not be desirable. See Figure 8.

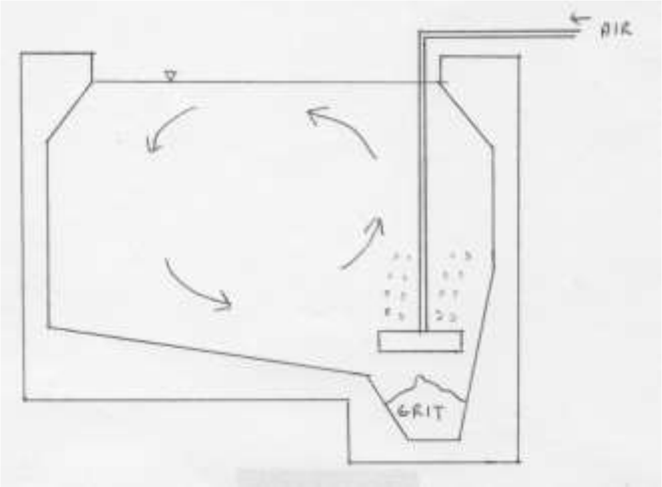
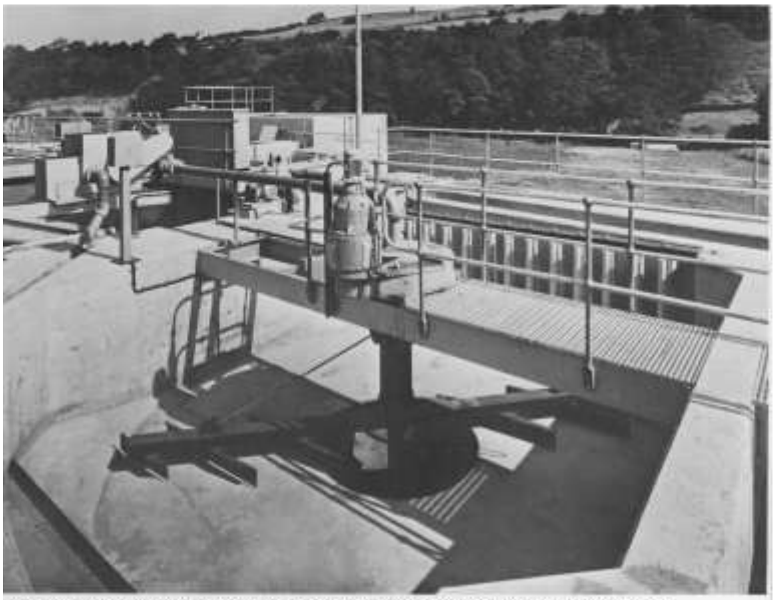


Figure 8 – AERATED GRIT CHANNEL.

3.6.5 The horizontal flow grit tank.

One make of this type is known as a "Detritor". It consists of a square tank of shallow depth, with the inlet channel angled to the direction of flow across the tank. A series of concrete deflection baffles, pivoted at the top and bottom, is fitted across the full width of the tank at the inlet end and these can be adjusted to maintain an even distribution of flow throughout the unit. A weir at the outlet end of the tank maintains the water level and retains the grit which settles on the flat bottom. The collecting mechanism is carried on supports spanning the tank, the floor of which is circular in form. Settled grit is moved continuously from the centre to the periphery, where it is collected in scoops attached to the ends of the rotating arms and is discharged into a sump. From the sump, grit is discharged into a cleansing channel attached to the tank. The floor of the cleansing channel forms a continuous ramp. The mechanism is also inclined and its reciprocating action, intermittently progresses the grit up the ramp towards the discharge point, the grit being freed from the organic matter by the washing action set up by the reciprocating rakes. This may be seen in figure 9.



A 5.0m diameter J & A Crossflow Grit extraction plant installed at Penistone Sewage Treatment Works. (Under construction for Yorkshire Water Authority—Southern Division) Consultants: A.H.S. Waters & Partners.

Figure 9 – HORIZONTAL FLOW GRIT TANK.

3.6.6 The circular flow grit chamber.

1. The vortex grit separator.

Wastewater enters the circular tank tangentially, organic matter being kept in suspension and eventually swept over the peripheral weir by the vortex influence. This action is assisted by an air diffuser or in the larger units by paddles. The settled grit is lifted from the hopper bottom of the tank by an air-lift pump and delivered to the adjacent classifier or washer unit. Several types of washer units may be used.

2. The Pista Grit Trap.

This is so designed that the power-driven paddles and "peripheral drag" maintain the correct horizontal velocity of the wastewater whilst it is flowing through the circular tank. The grit settles in a central sump and any organic matter which has settled with it is disturbed by a counter-current of air in the sump before the grit is removed. The wastewater lying above the grit may be displaced upwards by pumping final effluent into the grit chamber. This helps to clean the grit. The grit is then removed by an air-lift pump, allowed to drain and is then disposed of. This design gives satisfactory grit removal at all rates of flow and has the advantage that there are no moving parts in contact with the grit. See Figures 10 and 11.

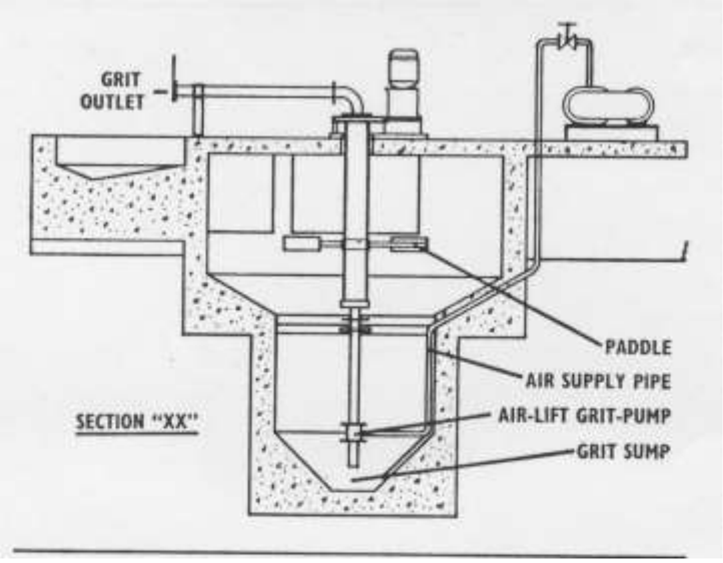


Figure 10 – PISTA GRIT TRAP - CROSS SECTION.

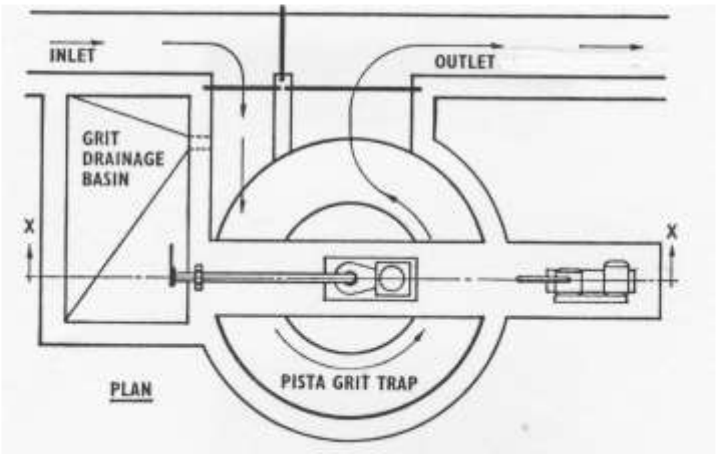


Figure 11 – PISTA GRIT TRAP – PLAN VIEW.

3.7 REMOVAL OF ORGANICS FROM THE GRIT.

The grit removed by the above processes will contain a significant quantity of organic matter. Some of it such as seeds, matches etc. are not a problem and may remain in the grit. Other organic material smells and needs to be removed. There are two main types of classifiers in use.

1. oscillating;
2. rotational.

Both processes move the material up an incline and allow the lighter organic material to be separated by turbulence and to be returned to the main wastewater stream. Both processes discharge about 1.5 metres above the water level and this allows some dewatering of the grit. The grit is then discharged into a storage hopper or a skip bin for transport off-site for disposal. This is usually in a landfill together with domestic and other waste. See figures 12 and 13.

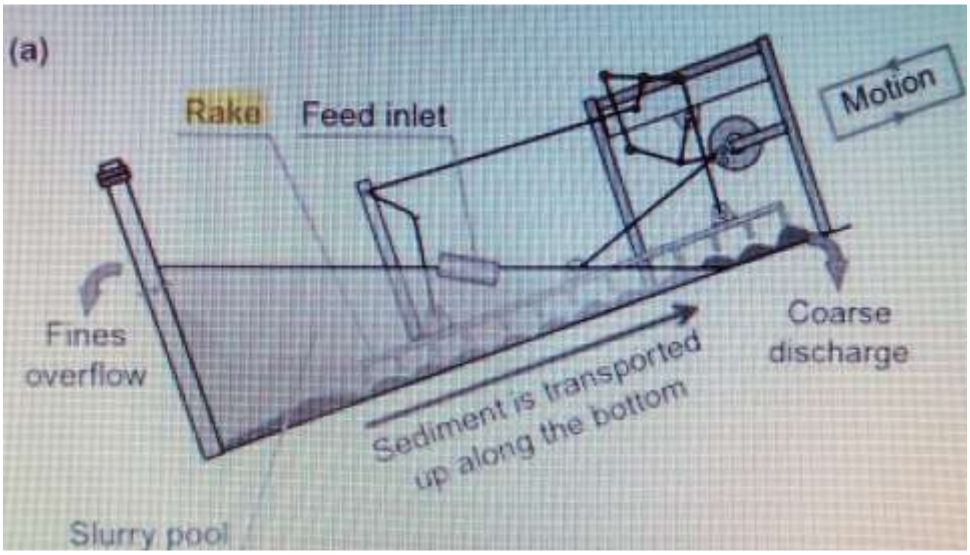


Figure 12 – OSCILLATING RAKE CLASSIFIER.

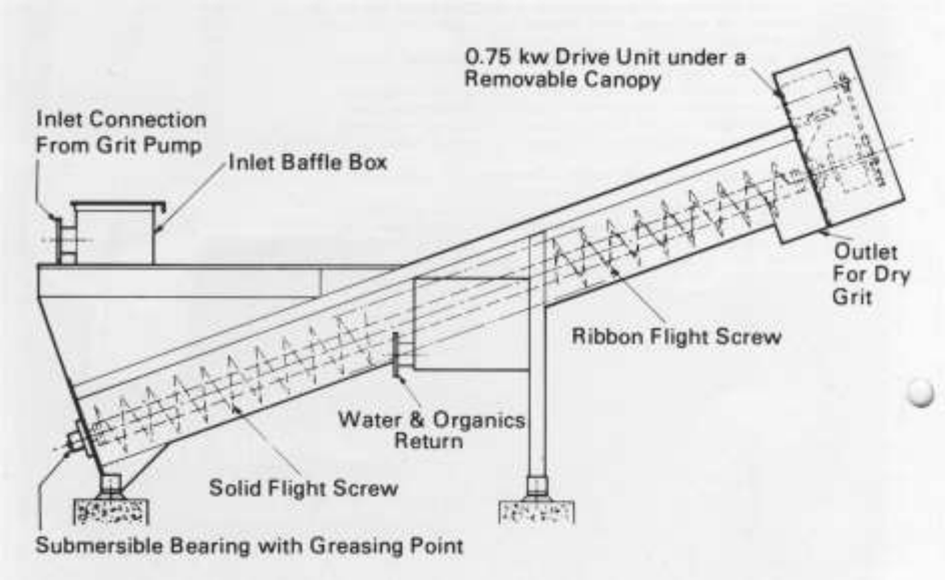


Figure 11 – SCREW CLASSIFIER.

3.8 FATS, OIL AND GREASE REMOVAL.

As a separate treatment stage, this process is rarely used these days. It is preferable to insist that those industries that could discharge these materials pre-treat their effluent before discharge to the sewer. A separate oil and grease removal stage would need to be sized to handle the peak flow reaching the treatment works.

The modern trend is to have a surface scraper on the primary sedimentation tank to remove any fats, oil, grease and other matters that rise to the surface.

3.9 DISPOSAL OF SCREENINGS AND GRIT.

There are two main methods in use:

3.9.1 Incineration.

This is the best method, but it is also the most costly. It has the advantage that the volume of the screenings, grit and grease is reduced to a small fraction of the former value. Fuel costs can be reduced by using digester gas. The higher the water content of the screenings, grit and grease, the more fuel is needed to boil off the water and so the more expensive the process becomes. There are several makes of screenings press available to squeeze water out of the screenings.

Where screenings, grit and grease are to be incinerated, then the rotary hearth incinerator is likely to be the most suitable. On large works where sludge incineration is also undertaken then the screenings and grit can be fed in with the sludge in a multiple hearth incinerator.

3.6.2 Burial.

On-site burial of screenings and grit is no longer permitted due to the high probability of pollution of groundwater. The screenings and grit may be co-disposed with domestic solid waste in a suitable solid waste disposal site.

WASTEWATER TREATMENT.

PART 4

PRIMARY TREATMENT

4.1 INTRODUCTION.

Wastewater is complex and contains polluting matter in many forms. In addition to the dissolved impurities, it contains a heterogeneous dispersion of numerous substances, both organic and inorganic, colloidal, pseudo-colloidal and simple suspension. Solid material and most of the large insoluble inorganic matter are removed by the preliminary processes mentioned before, but the remaining suspended matter, which is mainly organic and highly polluting, passes forward in the body of the liquid.

The purpose of sedimentation is to separate the wastewater into two main components, sludge and settled wastewater, which by being treated separately are normally dealt with more economically and efficiently. Generally about to 40% of the total pollutional load (measured as the Chemical Oxygen Demand) in the wastewater is removed by settling. About 60% of the Suspended Solids should be removed by primary sedimentation and nearly 100% of the settleable material..

The secondary treatment of wastewater results in the production of further organic solids (secondary sludges) which are more difficult than primary sludge to dewater. Thus any process that can reduce or minimize the production of secondary sludges is to be favoured.

If settleable solids are separated from wastewater by gravitation and by the natural coagulation and flocculation of the particles alone, the process is called plain sedimentation. If chemicals are added to induce, hasten or increase coagulation, the process is called chemical precipitation.

As seen later, treatment works using Biological Filters will always have primary sedimentation tanks. Activated Ssludge works can be designed to operate without the primary sedimentation stage. They are then operated in what is known as “extended aeration” mode. This is covered in greater detail later.

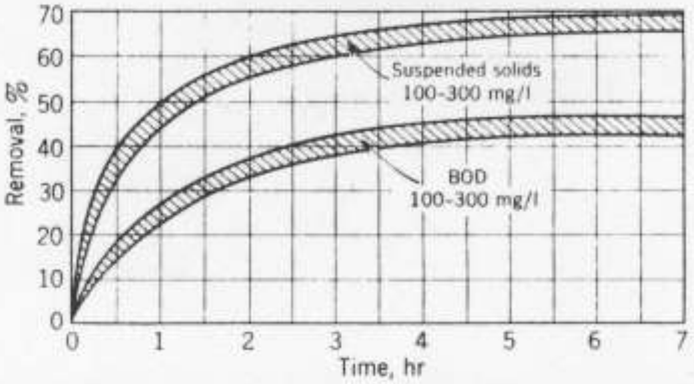
4.2 PLAIN SEDIMENTATION.

This is the process by which the velocity of the wastewater is reduced below the point at which it can transport the suspended matter, so that much of this settles and can be removed as sludge.

Basically, the purpose of sedimentation is to remove the maximum amount of polluting matter, in the form of readily settleable solids, from the wastewater as quickly, and as economically as possible. Sedimentation has some beneficial side effects:

1. flocculation of the finely dispersed solids and adsorption of colloidal and pseudo-colloidal matter onto the solids which may then settle,
2. because of the 2 to 4 hour retention time in the primary sedimentation tank, some equalization of the strength of the wastewater takes place prior to the secondary treatment.

Although the removal of sludge by the process of sedimentation reduces the volume of a normal wastewater by about 2% (i.e. the volume of sludge withdrawn from a primary sedimentation tank is about 2% of the volume passing through the tank), it reduces the strength or the Chemical Oxygen Demand by about 40 %. The sludge that settles usually contains about 5 % m/v dry solids. The removal of the Biochemical Oxygen Demand (BOD) and suspended solids by plain sedimentation; with time is shown in Figure 12.



Removal of suspended solids and biochemical oxygen demand from sewage by plain sedimentation in primary tanks.

Figure 12 – REMOVAL OF SUSPENDED SOLIDS AND BOD WITH TIME.

It is seen that beyond about 3 hours sedimentation time there is little to be gained by increasing the detention time.

TABLE 4 – REMOVALS AFTER 3 HOURS RETENTION TIME.

Settleable Solids	almost 100%
Suspended Solids	about 60%
Chemical Oxygen Demand	about 40%
Kjeldahl Nitrogen	About 10%

4.3 TYPES OF PRIMARY SEDIMENTATION TANKS.

The first primary sedimentation tanks were of the batch-wise or fill-and-draw type. These were usually square or rectangular and were usually manually desludged. The wastewater was discharged into the tank until it was full. The wastewater was allowed to settle and the water drawn off and the sludge removed. These are no longer used these days, having being replaced by continuous flow tanks.

4.3.1 Horizontal Flow Primary Sedimentation Tanks.

These are usually rectangular in plan, occasionally square tanks are used, with the floor sloping towards a sludge draw-off, at the inlet end. Although these tanks are called horizontal flow, the wastewater usually enters at a lower level than the invert of the outlet weir. The general direction is from one end of the tank to the other, with the heavier solids being deposited at the inlet end and progressively lighter solids being deposited towards the outlet end. These are not often used. One argument is that they have proportionally a much shorter weir than a similar sized round tank. This can be compensated for by having a double sided weir.

4.3.2 Radial Flow Primary Sedimentation Tanks.

These are the most commonly used type of tank. These are circular in plan, with the floor sloping at 7 to 15% to a central sump. The inlet is at the centre and since the level at which the wastewater enters the body of the tank is usually lower than that of the peripheral weir, the flow has an upward as well as a radial component. Also, during its passage from the central inlet to the peripheral outlet the velocity of the wastewater is constantly decreasing, reaching a minimum at the outlet weir.

4.3.3 Upward Flow Primary Sedimentation Tanks (Dortmund Tank).

These may be square or circular in plan and are basically an inverted pyramid or cone, with the floor sloping sharply to a central sump. The inlet is at the centre and the direction of movement of the wastewater when it enters the body of the tank is downward, it then flows radially outward and upward towards a peripheral outlet weir. The flow is more upward than outward, hence the name "upward flow" tank. These tanks are no longer used today due the great depth and hence cost of larger tanks.

A variation of the Dortmund tank is the two-storey or Imhoff tank, where the lower portion of the tank is used as a sludge digester. This is an old fashioned process that will be covered in greater detail when dealing with sludge treatment.

4.4 SLUDGE COLLECTION AND WITHDRAWAL.

4.4.1 Mechanical Sludge Scraping and Withdrawal in Horizontal Flow Tanks.

Here two main types of scrapers are used:

1. travelling bridge scrapers, where a power driven bridge spans the tank from which are supported the sludge scrapers. The speed of scraping is usually about 1.2 m/min on the forward journey and about 2.4 m/min on the return journey when the scrapers are either lifted or tilted. Formerly the bridges were rope hauled but problems occurred with crabbing. Among the modern methods used are rack and pinion drives. See Figure 13.

2. flight scrapers, where blades span the tank and are attached to endless chains, one on each side of the tank. The chains run on sprocket wheels supported from the side walls of the tank, two near the floor and two just below the water surface. The two chains are driven from a common shaft by an electric motor mounted on the tank wall at the inlet end. The scrapers move along the tank floor at about 0.6 m/min and sweep the sludge into a hopper or hoppers at the inlet end. As the scraper returns along the surface it sweeps the scum to a collection point at the outlet end of the tank. The fact that scum is swept to the outlet end of the tank can be a disadvantage. These are not often used as they have the problem of moving parts under water. These require frequent maintenance that can only be done by emptying the tank. See Figure 14.

In the horizontal flow tank, sludge is withdrawn under pressure due to a differential head. When hoppers are provided, there are usually two or more each with its own sludge draw off

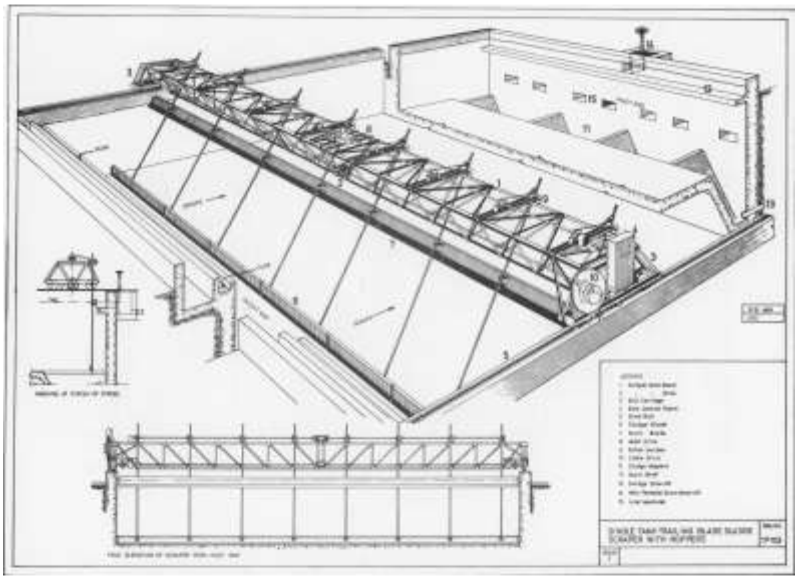


Figure 13 – RACK & PINION DRIVE SCRAPER.

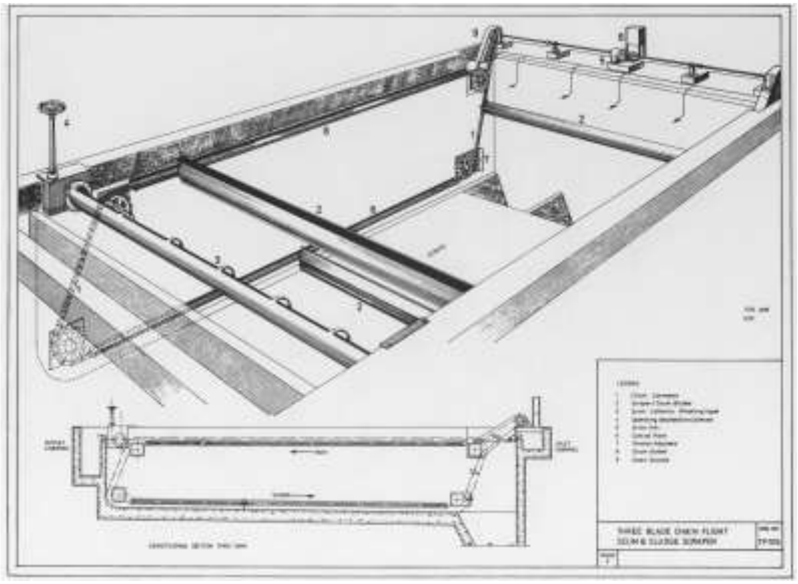


Figure 14 – CHAIN FLIGHT SCRAPER.

4.4.2 Mechanical sludge scraping and withdrawal in Radial Flow Tanks.

Two main types of scrapers are used:

1. Fixed bridge scrapers, where the bridge spans the tank and supports a walkway with hand railing providing access to the drive unit. The drive unit is mounted at the centre of the tank and consists of an electric motor driving a vertical shaft through reduction gearing. The shaft supports the two scraper arms and has a special watertight bearing at floor level.

   The feed pipe is usually laid beneath the floor to the centre of the tank and then turns upwards to discharge into a deflector drum. The electricity supply cable for the motor is carried along the bridge and this arrangement is much simpler than the power supply to a rotating-bridge scraper. With tanks up to 15 m diameter, the bridge structure is usually fabricated from channels or joists (RSJ's) suitably braced, whilst the lattice design is used for tanks of larger diameter.

2. Rotating bridge scrapers, here there are two sub-types:

   i half bridge, extending over the radius of the tank;

   ii. full bridge, extending over the diameter of the tank.

Occasionally 3 arm bridges are used on very large tanks. A full-bridge scraper is usually centrally supported and one end of the bridge carries the drive motor, the other end being supported on an idler wheel. A half-bridge scraper is pivoted at the centre and either driven from the centre or by a motor on the outer end of the bridge.

The electrical supply system is more complicated than with a fixed bridge scraper, the cable having to the laid in a duct in or under the tank floor and carried up the side of the feed pipe or in the central column. With smaller tanks, the cable may be carried overhead, being supported by a catenary wire. When laid in a plastic duct, modern methods of sealing have reduced the earlier hazards. In either case there is a slip-ring assembly at the tank centre, with a cable to the drive motor.

In the earlier design of scraper, a continuous helical blade was used but this was unable to automatically adjust itself to the profile of the floor if the screed finish was inaccurate. Blade arrangements now normally used in P.S.T.'s are either:

1. a series of blades arranged in echelon at an angle of about 45° to a rotating radial arm, the number depending on the diameter of the tank, See Figure 15 or;

2. a series of straight blade sections arranged to form a continuous volute. See figure 16.

With blades arranged in echelon the sweeping process is more gradual since, unless the blades overlap, the unit must make as many revolutions as there are blades to convey sludge from the periphery of the tank to the central hopper, although less load is imposed on the scraper mechanism. With the continuous volute, the sludge will be moved to the centre hopper much more quickly. This reduces the chances of gas production and the re-suspension of the sludge.

Scraper blades are often attached to hinged arms and are trailing, relying on their weight or on a spring-loaded action to prevent their being lifted from the floor by an accumulation of sludge. When on a hinged arm, the blade automatically adjusts itself to the profile of the tank floor.

Blades are edged with renewable rubber or rubber substitute wearing strips, some have a metallic edge. However, the weight of the trailing portion should not be such as to produce undue wear. Scraper blades are supported from a frame work carried from the bridge and if freely attached to the bridge structure they can be lifted for examination without having to empty the tank.

In primary sedimentation tanks, the peripheral speed of the scraper is usually in the range of 1.8 to 2.5 m/min.

An additional scraper is sometimes attached to the rotating framework carried by the bridge to maintain the cleanliness of the vertical side wall of the tank.

In the radial flow tank, sludge is withdrawn from the central hopper through a pipe running under the floor of the tank to an external inspection chamber. Here the rate of withdrawal can be controlled by a telescopic screw-down bell mouth weir or an adjustable straight weir.

In both horizontal flow and radial flow tanks, the sludge control valves may be hand operated, power operated by remote control or may even be partially or fully automated.

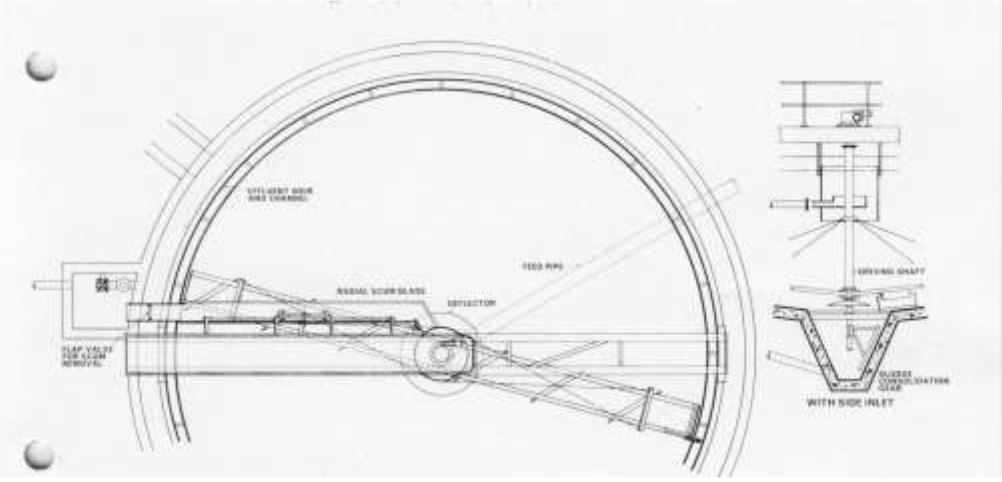


Figure 15 – SCRAPER BLADES IN ECHELON LAYOUT.

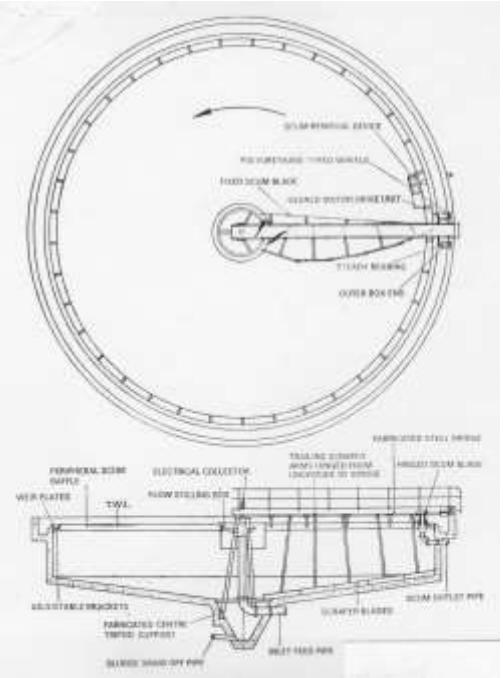


Figure 16 – SCRAPER BLADES IN CONTINUOUS VOLUTE LAYOUT.

4.4.3 Mechanical Sludge Scraping and Withdrawal in Vertical Flow Tanks.

It is usually not necessary to have scrapers in vertical flow tanks, since the sludge will slide down the steeply sloping sides, and will collect at the bottom. Sludge is withdrawn from the bottom through a pipe located inside the tank and terminating a short distance above water level. At least 1.2 m, and preferably 1.5 m below water level the pipe has a horizontal branch which discharges into an inspection chamber at the side of the tank. Sludge is withdrawn through this branch, whilst the vertical extension is used as a rodding "eye".

Sludge is withdrawn under pressure due to a differential head and discharges over a telescopic screw-down bell mouth weir or a swinging arm. The mouth of the sludge pipe should be 1.5 pipe diameters above the bottom of the tank to prevent blockage at this point, and if it is chambered so that it is in a vertical plane, this will facilitate both withdrawal of sludge and rodding, if this becomes necessary. The sludge pipe should be at least 150 mm diameter. See Figure 17.

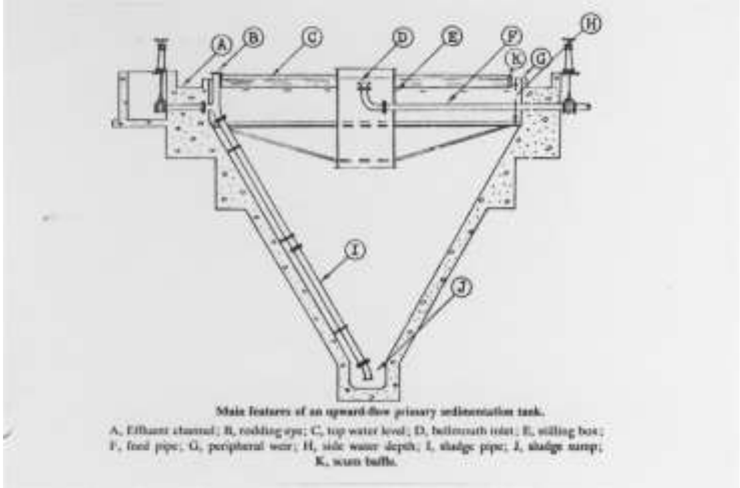


Figure 17 – UPWARD FLOW PRIMARY SEDIMENTATION TANK.

4.5 SCUM REMOVAL ON PRIMARY SEDIMENTATION TANKS

Where oil and grease removal has not been used as a preliminary treatment, it may be incorporated with the primary treatment stage. The mechanism for scum removal is usually incorporated in the mechanism for sludge scraping. The exact details vary according to the make of the unit.

4.6 SEDIMENTATION TANK LOADING PARAMETERS.

The hydraulic loading of sedimentation tanks (both Primary and Secondary) may be expressed in several ways:

4.6.1 Retention Period, also known as Detention Period.

This is the time taken for a particle of liquid to flow through the tank. This term must always be carefully used in that the retention period can be either the "theoretical" or the "effective" retention period. The theoretical retention period is the time taken for a particle of liquid to pass through the tank assuming that the flow is perfect, i.e. every particle of liquid is in the tank at the time that the labelled particle enters is discharged before the labelled particle in turn is discharged. This does not happen in practice and many particles often "short circuit" or take a short cut through the tank and pass through very quickly. Certain other particles follow devious routes and pass through the tank very slowly. The average time taken for a particle to pass through the tank is called the **effective** retention period.

The mean **theoretical** detention period in hours is calculated by dividing the volume of the tank (m^3) by the mean rate of flow per hour (m^3/h).

Example of theoretical retention period calculation:

Dimensions of primary sedimentation tank – 20 m diameter by 3m side wall depth.

Average flow rate	250 m^3/h (kl/h);
Peak flow rate	625 m^3/h. (assuming a peak wet weather factor of 2.5)

Volume of tank = 3.142*diameter*diameter*depth/4
= 3.142*20*20*3/4
= 943 m^3

Retention Period = 943 m^3 divided by 250 m^3/h
= 943/250 h
=3.8 h at average flow

Retention Period = 943 m^3 divided by 625 m^3/h
= 943/625 h
=1.5 h at peak wet weather flow

Effective detention time. Even in well-designed tanks, some of the inflow reaches the outlet weir in less than the theoretical detention period, and some takes much longer to do so. The degree of short-circuiting and the extent of retardation can be measured by adding a single but adequate amount of dye, electrolyte, or other tracer substance to the sedimentation tank influent and observing the rise in concentration of the substance reaching the outlet weir as time passes. Relating the observed times to the theoretical detention times permits the making of comparisons between different flow rates in the one tank and between different tanks.

4.7 DETERMINATION OF THE EFFECTIVE RETENTION TIME.

It was stated earlier that the effective retention time is always less than the theoretical retention time. It is important to know what the effective retention time is, both in time units and as a percentage of the theoretical retention time. Although there are many methods of determining the time, they all use the same basic principle of adding some chemical to the water and noting the time when it passes over the weir.

Among the chemicals that may be used are:

1. Sodium chloride, measurements may be made to measure the sodium content, the chloride content or the conductivity;

2. Lithium chloride, here a flame photometer or atomic absorption spectrophotometer would be needed;

3. a dye such as Sodium Fluorescein or Rhodamine B. Here a photometer or colourimeter would be needed;

In this example, sodium chloride is used. The principle of the test is based on the rapid addition of the salt to the influent of the sedimentation tank under study and the determination of the concentration of the salt in the effluent at frequent intervals. The first appearance of a higher chloride content in the effluent represents the time of arrival of a portion of the chemically treated influent. This may occur in 10-20 minutes, but a portion of the chemically treated influent will be found to be leaving the tank long after the lapse of the theoretical detention period. Fluctuations in the concentration of salt in the effluent disclose erratic flow or short-circuiting portions of the influent. The salt concentration is determined by measuring the chloride or the sodium content of the influent and effluent.

In short, the procedure is add enough salt to give the effluent a chloride content of about 300 mg/L Chloride for 1.0 minutes. If the flow is 20 L/s, then in 1 minute 20 * 60 or 1 200 litres will pass the influent reference point. The chloride content must be 300 mg/L as chloride. Using sodium chloride which contains 60% chloride, the mass of sodium chloride needed is 360/0.6 = 600 g sodium chloride. This NaCl must be dissolved in 20L of water and this solution must be poured into the effluent at a point where good mixing is obtained.

Add the 20L as quickly as possible. Immediately start taking samples of influent and effluent, the first influent sample must be taken at the instant that the salt is added. Calling the first sample of both influent and effluent; 0 mins, the samples should be taken at 1, 2, 5, 10, 15, 20, 30, 40, 60, 80, 100, 120, 150, 180, 210, 240, 300, 360, 480 minutes. Determine the chloride or sodium content of each sample and then plot the chloride or sodium content vs time on linear/linear paper. Plot the chloride or sodium on y-axis and time on x-axis. Draw the curve through the points representing the influent samples and the curve through the points representing the effluent samples. This may be seen in figure 18.

In this case the sodium chloride was added some distance upstream of the influent sampling point, this is shown by the fact that the sudden short-term increase in influent chloride took place 10 minutes after the sodium chloride had been added.

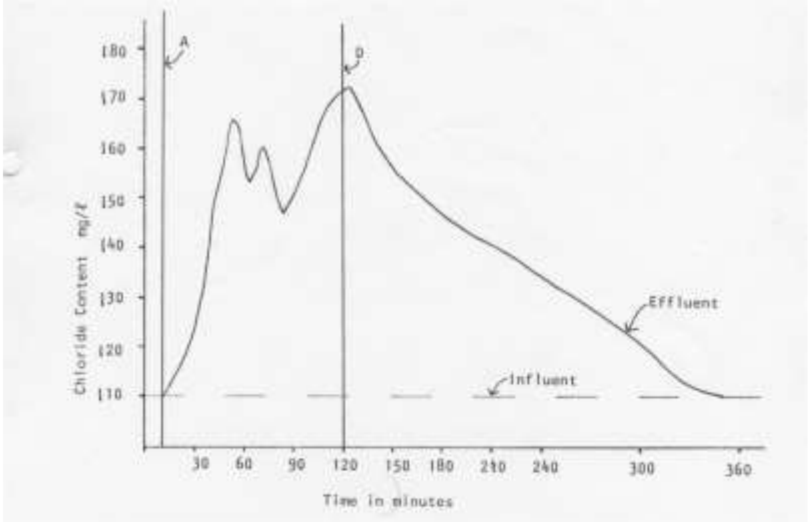


Figure 18 – DETERMINATION OF EFFECTIVE RETENTION TIME.

The area between the influent and effluent curves is cut out with a pair of scissors and the centre of gravity of this area is determined. One way to do this is by carefully balancing the piece of paper on a level knife edge so that the knife edge is parallel to the Y-axis. In this case, line D passes through the centre of gravity. The area under the curve on the left of line is the same as to the right of line D.

This line represents the time at which exactly 50 % of the mass of sodium chloride had passed through and out of the tank. The effective detention time is then the time interval between the time when the high chloride water entered the tank (line A), and time when 50 % of the chloride had left the tank (line D).

Assuming that the theoretical detention time was 200 minutes, line A represented 10 mins and line D 120 minutes. In this case, the influent chloride content was 10 mg/L.

$$\text{The efficiency of displacement} = \frac{\text{effective detention time} * 100}{\text{theoretical detention time}}$$

$$\text{In this example} = \frac{120-10}{200} * 100 = 55\ \%$$

Values of 5 - 30 % represent poor tanks
30 - 50 % represent good tanks
50 - 60 % represent very good tanks
> 60 % represent excellent tanks.

Figure 19 shows the shape of the curve that would be obtained in various tank types.

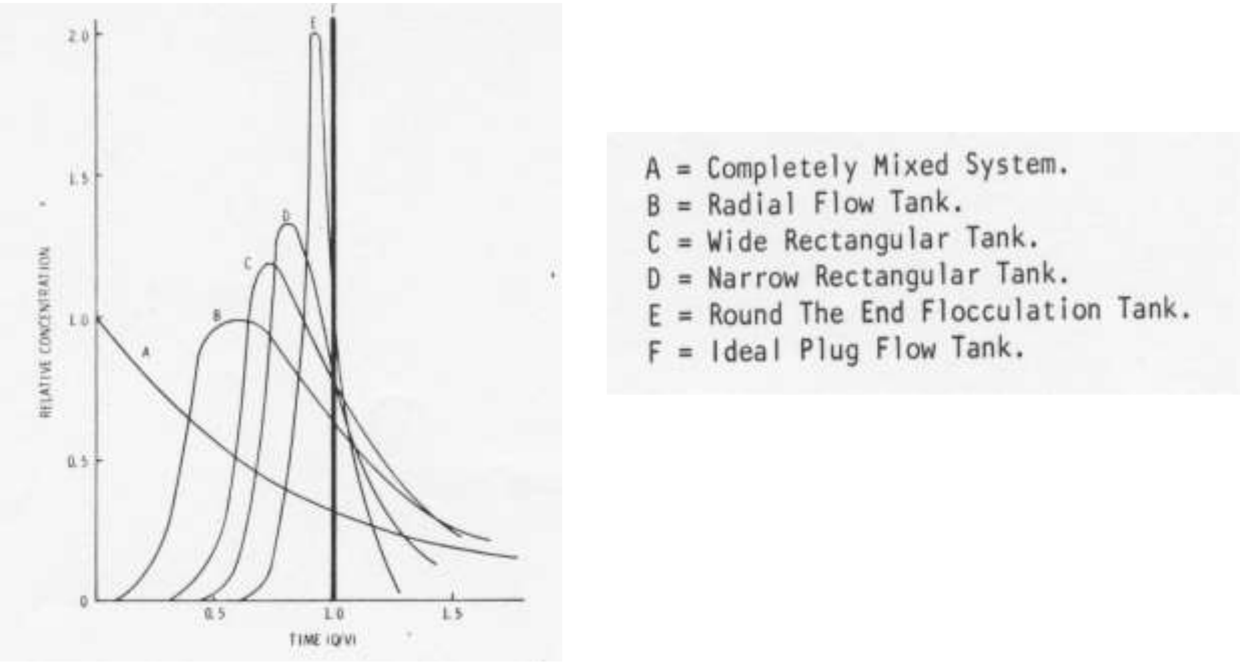


Figure 19 – EXAMPLES OF EFFECTIVE RETENTION TIME.

4.8 SURFACE LOADING RATE OR UPWARD VELOCITY.

The latter is expressed at m/h. The former is expressed as m^3/m^2-h, but this is seen to be dimensionally equivalent to the upward velocity. This is calculated by dividing the flow rate in m^3/h by the surface area of the tank in m^2. This value is equal to the theoretical vertical velocity of flow by which the liquid, under perfect conditions, approaches the surface for draw off. This rate is important in relation to the settling velocity of the suspended solids, since, if this upward velocity is greater than the settling velocity of a particular particle, then that particle cannot settle and it will be carried upwards and over the weir.

Example of calculation of Surface Loading Rate**.**

Using the above example, the cross sectional area (XSA) of the 20 metre tank is 3.142*20*20/4 = 314m².

The average surface loading rate = 250/314 = 0.80 m³/m²h or 0.80 m/h.

The peak surface loading rate = 625/314 = 2.0 m/h.

4.9 WEIR OVERFLOW RATE.

This is a measure of the liquid load passing over the discharge weir. It is calculated by dividing the mean daily flow by the total length of weir. Again mean and peak values may be calculated. It is expressed **in m³/m-d.**

If the maximum flow rate over a weir is less than about 4 m³/m-h, problems may occur because of surface tension effects. If the weir overflow rate exceeds 19 m³/m-h at maximum flow, the velocity of the wastewater as it approaches the weir, will be excessive and sludge may be scoured out of the tank and scum may be drawn under the scum baffle. In this case, double sided weirs may be necessary.

Example of calculation of Weir Overflow Rate.

Using the above example, the weir length will be 3.142*20 = 62.8 m.

The mean overflow rate is 250 m³/h / 62.8m = 4 m³/m-h;

The peak overflow rate is 625 m³/h / 62.8m = 10 m³/m-h.

4.10 CHEMICAL PRECIPITATION AS A PRIMARY TREATMENT PROCESS.

This is similar to the process used in water treatment, but usually larger dosages of chemicals are used. Colloidal solids and very finely divided suspended matter cannot be removed from wastewater by plain sedimentation unless they are rendered settleable by the addition of coagulating chemicals. Usually biological action in the secondary treatment stages is sufficient to render the matter settleable in the secondary sedimentation tank. In the cases of hydraulic or organic overload, the faster settling rate of the coagulated solids can help relieve a hydraulic overload condition and the increased suspended solids removal can help relieve an organic overload condition.

The coagulants common in chemical precipitation react with substances present in the wastewater or added for this purpose:

1. to form relatively insoluble coagulant flocs that enmesh or adsorb suspended or colloidal particles with which they come into contact;

2. to form coagulant colloids that are attracted to and coalesce with oppositely charged wastewater colloids to form settleable flocs.

Chemicals used for chemical precipitation of wastewater include:

1. Lime CaO or $Ca(OH)_2$
2. Ferrous Sulphate $FeSO_4$ H_2O
3. Ferric Chloride $FeCl_3$
4. Ferric Sulphate $Fe_2(SO_4)_3$. $3H_2O$
5. Aluminium Sulphate $Al_2(SO_4)_3$ $14H_2O$

The chemical would be dosed upstream of the primary sedimentation tank where there is good mixing. Actual dosages needed are determined experimentally.

The return of chemically precipitated sludge to the primary sedimentation tank or of a portion of this sludge to the influent to chemical precipitation works for the purpose of improving clarification or producing a denser sludge seems to be of doubtful economy. In the treatment of industrial wastes, on the other hand, return of sludge may be worthwhile.

The volume of sludge produced by chemical precipitation is large:

1. because greater amounts of solids are removed;

2. because the precipitants themselves form part of the sludge;

3. because the water content of the sludge is high.

Almost twice the volume of sludge is collected as by plain sedimentation if lime is employed as precipitant alone. In combination with iron salts even greater volumes accumulate. Chemical sludges digest easily and are amenable to dewatering.

As a treatment process, chemical precipitation accomplishes removals of solids and COD about midway between plain sedimentation and complete biological treatment. Removal of about 80 % of the suspended solids and 65 % of the COD are generally recorded for chemical dosages of about 30 mg/L of ferric chloride. If the amount of chemical is doubled, removals as high as 90 % of the suspended solids and 75 % of the COD may be removed.

Within the limits of the process, one of its attractive features is its flexibility. Efficiencies can be varied as needed. Seasonal use for the protection of streams during periods of low run off, or the safe guarding of bathing beaches and other recreational facilities during the summer months, does not require much stand-by equipment. Only small portion of the plant viz., the chemical storage, handling and mixing sections, is taken into or out of use. In full-time operation however, chemical treatment is less efficient and as least as expensive as full biological treatment.

Chemical precipitation lends itself to the treatment of various industrial wastes and of the wastewater of industrial cities in which toxic or other trade wastes endanger the operation of biological treatment works or render biological processes inactive. In spite of these advantages the process is very seldom used.

4.11 OPERATION OF PRIMARY SEDIMENTATION TANKS - DESLUDGING.

This is perhaps the most important and exacting operation in the running of a treatment works. The proper carrying out of the function depends on the proper operation of both the secondary treatment and the sludge handling processes.

Insufficient desludging will result in the sludge belching, that is rising to the surface, as a result of gas generation taking place in the sludge, with the resultant carry over into the secondary treatment unit. In biological filtration process, this results in overloading and can result in blockages in the medium. In the activated sludge process, this can result in the dissolved oxygen content dropping below the required level for nitrification.

Too much desludging will result in too much water being passed to the downstream sludge handling processes. As most digesters operate at 32° to 35°, this means that a much larger quantity of water must be heated from ambient to this operating temperature. The extra volume of water also reduces the sludge digestion period, if no supernatant removal is made from the digester.

In most of the sedimentation tanks in this country, the sludge is drawn off through a sludge pipe which is commonly controlled by a valve situated in an adjacent manhole. The sludge is discharged by hydrostatic pressure from the tank i.e. it is forced up the sludge pipe from the bottom of the tank to the level of the discharge valve by the pressure of the liquid in the tank.

Sludge must usually be discharged a minimum of two times per day, the draw-off times being arranged so that half the daily quantity of sludge is, as near as possible, discharged at each desludging. Care must be taken to ensure that the full quantity of sludge is drawn off and that the valve is closed immediately of it is seen that water is being removed instead of sludge. It is possible, if the hydrostatic head is high (2 m or more) and desludging has not been carried out for some time, that opening the valve fully will result in a funnel being formed in the sludge body in the tank through which clear water may be drawn, leaving the sludge in the hopper. To avoid this, it is important that the operating staff make themselves familiar with the quantity of sludge expected at each desludging. For example, it may be found that desludging is complete in 15 minutes on Monday morning, 10 minutes on Monday afternoon and so on.

Any gradual or sudden change in the quantity of sludge being removed, time to desludge etc. should be noted and the reason sought and found.

4.12 OPERATION OF PRIMARY SEDIMENTATION TANKS - SCUM REMOVAL.

Scum is usually removed through a scum trough or box. In mechanically desludged tanks this operation is usually automatic. The object of this process is to carry the floating scum to the scum trough and to draw it off with as little water as possible and with as little disturbance to the tank contents as possible. After removal, the scum is usually added to the digester.

4.13 DO'S AND DON’TS OF OPERATION.

1. do not allow any tools or other objects to fall into a sedimentation tank. A hammer jammed under a scraper can cause damage to the bridge mechanism;

2 do not allow untrained staff to desludge the tank without proper supervision;

3. do make sure that any person working over a tank is wearing a safety belt attached to a strong line and that a second person is always present, so that anyone falling into the tank may be quickly rescued;

4. do not disturb the water when cleaning weirs etc;

5. do make a habit of inspecting the sedimentation tank regularly each shift and look out for colour variation, belching, excessive scum etc. A great deal of trouble on the biological side can be avoided by such a careful watch.

4.14 PHYSICAL STRAINING PROCESSES.

Some treatment works use some sort of physical straining process either to replace primary sedimentation completely or by diverting peak flows through it to reduce the organic load on the secondary treatment process. There are two main physical straining devices.

4.14.1 Wedge Wire Screens

One example is the Bauer Hydra sieve which consists of three sections with successively flatter slopes on the lower sections. The screen wires are triangular in section and are spaced about 1.5 mm apart. The wires are usually made of stainless steel.

Although there is a fairly high head loss across the unit (up to 1 metre), some degree of aeration takes place at the screen.

A modification of the wedge wire screen is the rotating unit. The solids are trapped on the outside of the drum and are scrapped off.

4.14.2 Micro screening.

This is similar to the rotating wire screen but a fine woven material covers the outside of the drum. However solids are usually trapped on the inside of the drum. The mesh size is usually in the range 15 to 60 microns i.e. 0.015 to 0.06 mm.

WASTEWATER TREATMENT.

PART 5

SECONDARY TREATMENT PROCESSES.

5.1 INTRODUCTION.

The effluent from the primary treatment sludge will contain about 60% of the influent pollutional load, measured as the chemical oxygen demand. Virtually all the settleable solids have been removed, but about 40% of the influent suspended solids will remain. The effluent will be turbid due to the presence of colloidal material. At this stage, the total nitrogen removal is about 10 to 15%.

Under certain conditions, the effluent from the primary treatment stage may be discharged into the sea. In most cases, however, a greater degree of treatment will be required.

In secondary treatment, three processes will be described.

1. Biological Filtration;

2. Activated Sludge;

3. Rotating Biological Contactor.

Firstly some insight into the elements of biological treatment will be given, as the part played by micro-organisms is vital to the successful treatment of wastewater in the secondary treatment phase.

In nature, micro-organisms in their search for food breakdown complex organic materials into simple, more stable substances. The high energy organic wastes in wastewater are a rich source of food. These will support a host of living organisms with whose life activities are associated the self-purification of wastewater by so-called biological processes.

Whenever wastewater touches a suitable contact surface, there develops, at the interface between the wastewater and the contact medium or within the flocs, a growth of bacteria and other micro-organisms. These soon create slimy, gelatinous, zoogleal films or flocs which teem with unicellular and more complex organisms and possess the remarkable power of:

1. transferring to themselves matters that previously were held in suspension in the wastewater, in the colloidal state, and even in solution;

2. abstracting from the transferred substances the energy or nutrient necessary for film or floc maintenance and growth;

3. transferring back to the liquid the end products of decomposition, such as nitrates, carbon dioxide and sulphates. Just how this transfer is effected, is not known in detail. It is ascribed to adsorption, colloidal precipitation and biological precipitation including enzymatic actions.

Whatever the individual operations of contact action may be, the work of purification is assisted by the life activities of the micro-organisms that constitute, create, and maintains the interfaces. Their well-being is essential to the operation. Their needs include in particular a constant provision of food, an adequate supply of oxygen and a suitable supporting film or floc.

Bacteria and other organisms associated with them in biological treatment feed continuously, they need no rest. For best results, food should be furnished in sufficient amount to maintain at maximum activity a maximum biological population on each unit area of film or floc. Within limits, probably seldom exceeded in wastewater treatment, biological activity is accelerated by increased concentration or assimilability of food. Adsorption and other interfacial phenomena likewise are promoted by high concentrations of substances in flowing wastewater. For these reasons, the work done is greatest in the upper strata of biological filters and during the first hours of activated sludge treatment. Hence also, increased organic loading does not of itself reduce treatment efficiency until very high concentrations of waste matters are reached.

The decomposition of organic matters proceeds most rapidly when free or dissolved oxygen is available to the responsible micro-organisms. At normal temperatures, the rate of decomposition of wastewater is almost 3 times as great in the presence of dissolved oxygen i.e., aerobic decomposition, as in its absence i.e. anaerobic decomposition. The oxygen requirements of the active organisms are so high that the normal dissolved oxygen resources of wastewater, even if the wastewater is initially saturated with oxygen, are inadequate. The oxygen supply can be maintained at a satisfactory level only by exposing the surface films to air or by replenishing the oxygen supply of the liquid. So long as there is free oxygen, aerobic conditions will prevail, the actual dissolved oxygen content of the liquid appears to be of little consequence provided it is above the critical minimum content.

In biological filters, oxygen is furnished by air currents that sweep upward or downward through the filter whenever there is a reasonable difference in air and wastewater temperature. Ventilation is promoted by suitable bed construction.

The sludge flocs of activated sludge units are supplied with oxygen either from compressed air in diffused air units or from the atmosphere above surface aerators.

Oxygen requirements are reflected by the COD satisfied during treatment. This equals to COD applied to the biological treatment unit less the sum of

1. the COD transferred to the sludge and still remaining in it when it is removed from the system and;

2. the COD discharged with the effluent.

Expressed as a percentage of the COD applied, the COD satisfied during treatment is estimated to be:

1. about 45% in high rate activated sludge;

2. about 55% in high rate biological filters;

3. about 70% in low rate activated sludge;

4. about 80% in low rate biological filters.

These values are indirect measures of the relative stability of the sludges produced. They are almost independent of the wastewater treatment efficiency, which does not consider sludge stability but only relates the concentration of waste matters in the treated effluent to that in the influent wastewater.

The COD removal by biological contact processes approaches 90%. At normal summer temperatures, this value is reached in the self-purification of polluted streams only after about 10 days of flow. Wastewater is held in activated sludge tanks only for a few hours and percolates through biological filters in a few minutes.

However, COD reduction in rapid streams is due to actual biological decomposition or stabilization of the putrescible substances, whereas in biological treatment only a portion of the organic matter that can exert a COD, is decomposed. The remainder is transferred to the contact medium and eventually discharged as sludge.

Since the principle function of biological treatment is to convert non-settleable (finely divided, colloidal and dissolved) substances into settleable sludges, biological treatment units must be followed by settling tanks which remove sludge built up and unloaded by the biological units. Secondary sedimentation tanks are needed for this purpose. If the treated effluent from the treatment works is to possess the highest degree of clarity, then tertiary treatment is essential.

Of the organisms associated with biological wastewater treatment and with the self-purification of streams, the filamentous and unicellular (higher and lower) bacteria are the principal and most numerous biological workmen. These plant-like organisms surround themselves with a gelatinous, or zoogleal, slime. They live upon the organic matter and mineral substances in wastewater and convert them into the living cells within the active surfaces.

Closely associated with them are protozoa, among which one group, viz the ciliates, feeds upon bacteria. A primary function of these organisms is to prevent a stalemate in the bacterial population. Destruction of the bacteria leads to a constant renewal of bacterial growth and utilization of food substances. The prevalence of ciliates in the active slimes, particularly in activated sludge, is a valuable index of its condition. A large number of lively ciliates are observed when the activity of the film or floc is satisfactory.

Of secondary importance in the clarification of wastewater, but of great hygienic significance, is the function of protozoans in removing bacteria from the flowing wastewater. Disease-producing bacteria (pathogens) as well as bacteria that live upon dead organic matter (saprophytes) are devoured.

The biological associations include many other varieties of organisms. Worms and insect larvae find nutriment in the film and floc. Rotifers and crustaceans are scavengers of organic matter. Algae and fungi utilize simple chemical substances liberated by decay. Algae are small green plants that flourish only in the presence of sunlight, hence only on the surface of filters or tanks. In polluted waters, they are confined to the surface until sufficient clarification admits sunlight to greater depths. Then they may assist purification by release of oxygen as well as by abstracting nutrients from the water.

The organisms responsible for biological wastewater treatment have their origin in the water and in the waste matters that make up the wastewater. Hence biological activity establishes itself spontaneously. A certain time will elapse, before films and flocs are fully matured and normal efficiency of treatment is reached. As would be expected, this period is shorter in summer. Biological filters mature in a few weeks in summer. Normal operation in the activated sludge process can be hastened by seeding the works with sludge from another works. Carbonaceous oxidation can be well established in about 2 weeks and full nitrification can be established within a month at about 20°C.

WASTEWATER TREATMENT.

PART 6

BIOLOGICAL FILTERS.

6.1 INTRODUCTION.

This is known as a fixed medium system because the biological growth film is fixed onto some medium. In an established filter, it takes less than 30 minutes for most of the settled wastewater sprinkled on the surface to pass through a filter. This does not mean that all the settled wastewater has been purified in such a short time. Dissolved matter and the fine suspended materials are adsorbed onto the zoogleal mass and are broken down and oxidised over a period of hours or longer. Although the water may pass through the filter quickly, most of the humus that flows away with it has been in the bed for a much longer time.

Biological filters are either circular or rectangular in shape and usually have rotating or travelling distributors. Some designs have fixed sprays, to distribute the wastewater over the surface. The depth of the filter may vary between 1 and 3 m. In the deeper beds it is often necessary to have forced ventilation to ensure an adequate flow of air through the medium. The outer walls of the beds are usually made of concrete or brick, but walls of packed stone are sometimes used.

Crushed stone is the most commonly used as media for filling biological filters, but other materials such as coal, blast furnace slag, broken bricks and certain forms of PVC may also be used. The plastic medium has the advantage of being lightweight and having a much larger surface area per unit volume, but is more expensive. The main requirements for a suitable medium are:

1. it must be resistant to corrosion and erosion by the wastewater;
2. it must be strong enough to support its own mass;
3. it must be weather resistant;
4. it must be uniform in size;
5. it must not be so smooth that the zooglea cannot hold on;
6. it must have a good surface area to volume ratio, particularly for high rate filters;

The size of the media is usually 25 to 75 mm. Uniformity of size is important to produce maximum void space for adequate ventilation and drainage. If the media is not uniform in size, the smaller pieces fill up the voids between the larger ones, resulting in decreased performance of the bed, and even to choking with consequent surface ponding by the settled wastewater. The typical layout of a biological filter is shown in figures 21 and 22.

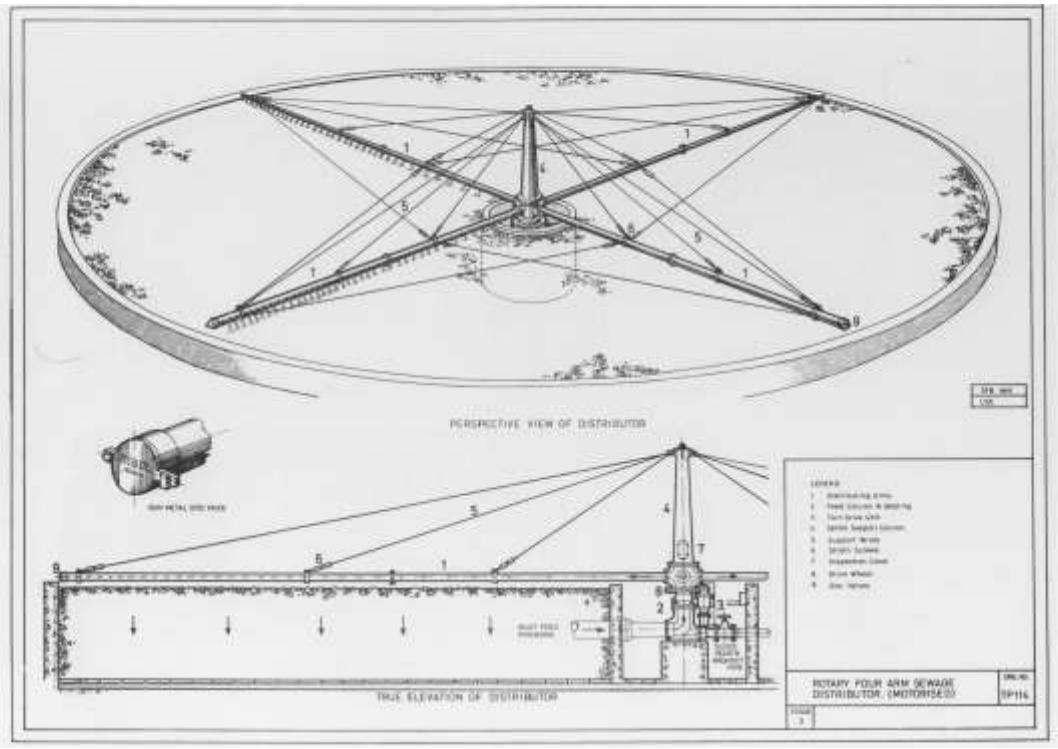
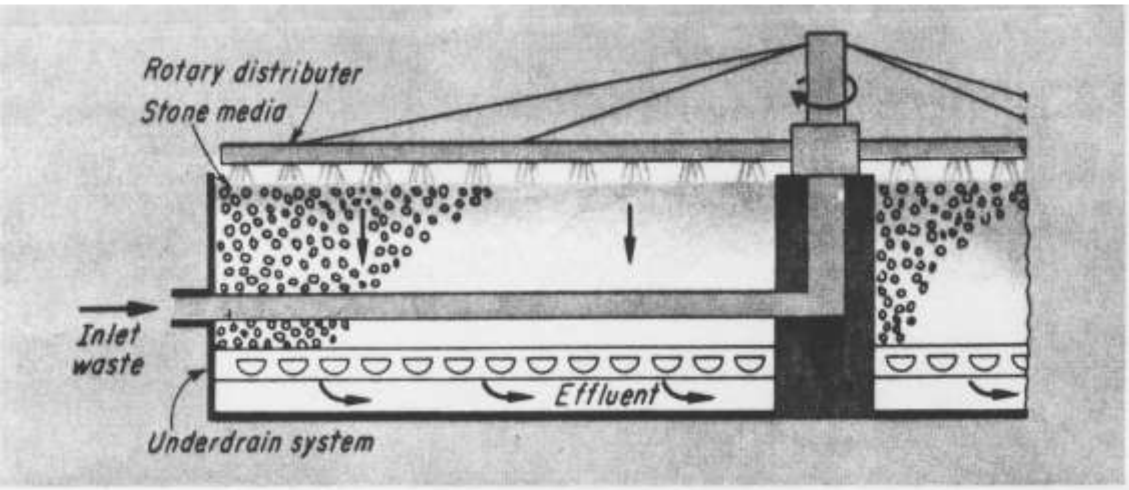


Figure 21 – BIOLOGICAL FILTER.



BIOLOGICAL FILTER consists of a circular basin filled with crushed rock, stone, slag or plastic. A rotating distributor mechanism spreads waste water over the filter bed

Figure 22 – Biological Filter showing more detail.

The wastewater passing down through the biological filter carries the food for the organisms attached to the filter medium while the air passing upwards provides the oxygen for the organisms. This is shown in figure 23.

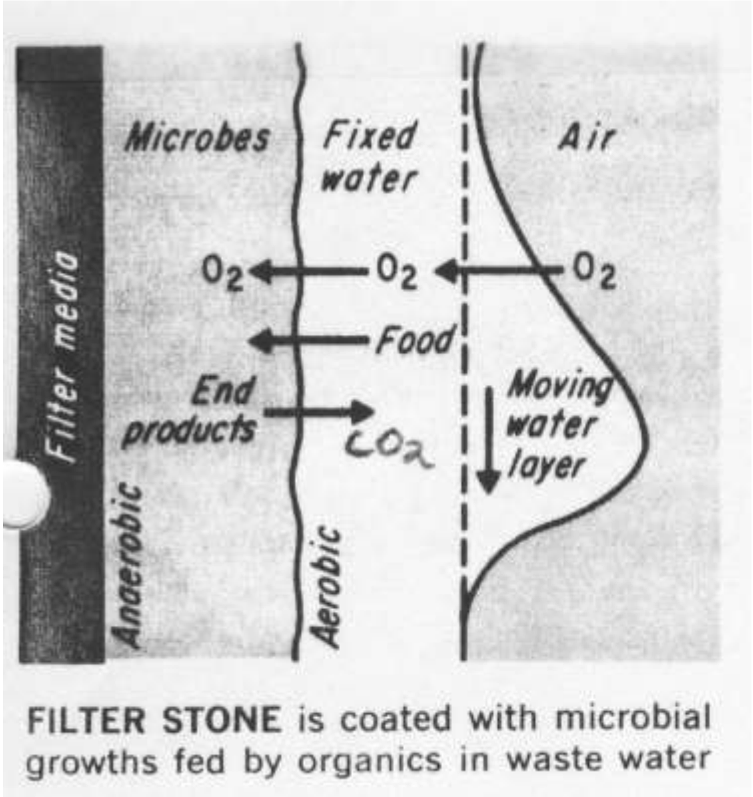


Figure 23 – EXCHANGES OF NUTRIENTS, WASTE PRODUCTS AND OXYGEN WITHIN A BIOLOGICAL FILTER.

The underdrainage system of biological filters normally serves two purposes:

1. to collect the effluent containing the humus solids and to discharge them from the filter;

2. to permit the free passage of air either upward or downward through the media.

Various patterns of underdrainage are in use. A common method is to the sloping floor with rows of half-round tiles with their ends open to the atmosphere at the outer end of each row.

In properly designed biological filters in which the media is of uniform size, natural ventilation is normally sufficient to supply the necessary air to the zoogleal mass. The natural ventilation is caused partly by the difference in temperature between the air and the wastewater, partly by wind, and also by air being sucked down by the wastewater as it drips from stone to stone downwards through the bed. There are two methods of running wastewater onto a filter:

1. by feeding the wastewater continuously over the whole surface all the time;

2. by feeding the wastewater so that individual areas are dosed intermittently.

In the first system fixed sprays or distributors are used, while in the second travelling or rotating distributors are used. Rotating distributors used on circular filters require 450 to 750 mm of hydrostatic head, and usually operate by the reaction of the wastewater leaving the orifices, but some may be motor driven. Travelling distributors are either motor driven or propelled by a water wheel which is operated by the flowing wastewater. In all cases, the openings or orifices of the distributors should be so spaced and dimensioned as to deliver a uniform amount of wastewater to all parts of the bed.

6.2 CLASSIFICATION OF BIOLOGICAL FILTERS ON THE BASIS OF LAYOUT AND METHOD OF OPERATION.

1. Single Stage Filtration;
2. Single Stage Filtration with recirculation;
3. Two stage Filtration with fixed sequence;
4. Two stage Filtration with alternating sequence.

6.2.1 Single Stage Filtration.

This is the simplest and most commonly used layout and is shown diagrammatically in figure 24a.

6.2.2 Single Stage Filtration with recirculation of settled effluent.

The use of recirculant has three advantages:

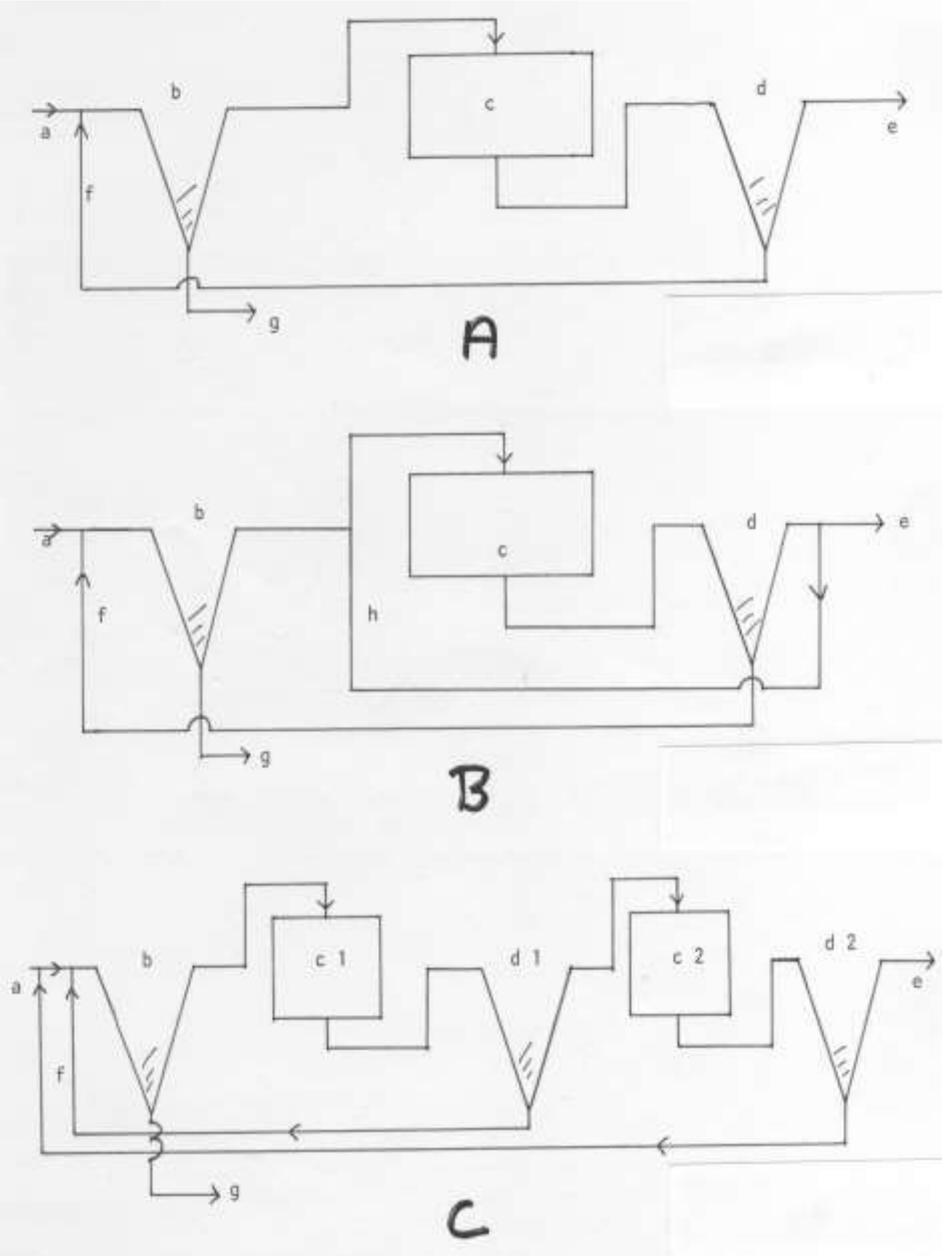
1. it reduces the strength of the wastewater being applied to the filter, this is important when high strength wastes are being treated;
2. by increasing the recirculant at low flow periods, it can be assured that the distributors are kept moving all the time. The presence of recirculant lowers the film thickness and reduces fly breeding by forced film sloughing;
3. if the filter is a low rate filter and is nitrifying the return of humus tank effluent containing nitrates to the primary sedimentation tank effluent tank effluent will result in the freshening of the settled wastewater and will result in a degree of denitrification taking place.

The ratio of the recirculated flow to the wastewater flow is the recirculation ratio. See figure 24b.

6.2.3 Two stage Filtration with fixed sequence.

In this system, the wastewaters passes from the primary filter into a humus tank, into the secondary filter and then into a secondary sedimentation tank. In this system it is common to use a coarser media in the primary filter than in the secondary. The secondary filter is often loaded at a lower rate. In this way the primary filter is responsible for most of the COD removal, while the secondary filter is responsible for most of the nitrification.

In this system, twice as much wastewater may be treated to the same standard than if the same volume of media was used in a single stage filter. See figure 24c.



a = raw feed ; b = primary sed. tank ; c = biological filter ; d = secondary sed. tank (humus tank) ; e = treated effluent ; f = sludge wasting into p.s.t. ; g = wasting of mixed sludges ; h = recirculation.

1 = primary ; 2 = secondary unit.

Figure 24 – VARIOUS BIOLOGICAL FILTRATION LAYOUTS.

6.2.4 Two Stage Filtration with alternating sequence.

In this system, settled wastewater is passed through two filters in series, first through a primary filter and then through a secondary filter with the filters usually fairly heavily loaded. The order of the two filters is alternated at intervals varying from one to seven days, the secondary filter then becoming the primary filter and the primary filter becoming the secondary, and so on. Under these conditions the growth of biological film which forms in the primary filter is loosened and washed out when this filter becomes the secondary unit, and this occurs with each alternation. The overall growth of this film is thereby limited. For this reason the system can deal with a heavier loading than with the other layouts.

This process is particularly suitable where large volumes of certain industrial wastes such as those from the dairy industry, tend to cause ponding on the surface of filters due to excessive growth of biological film. See figure 25

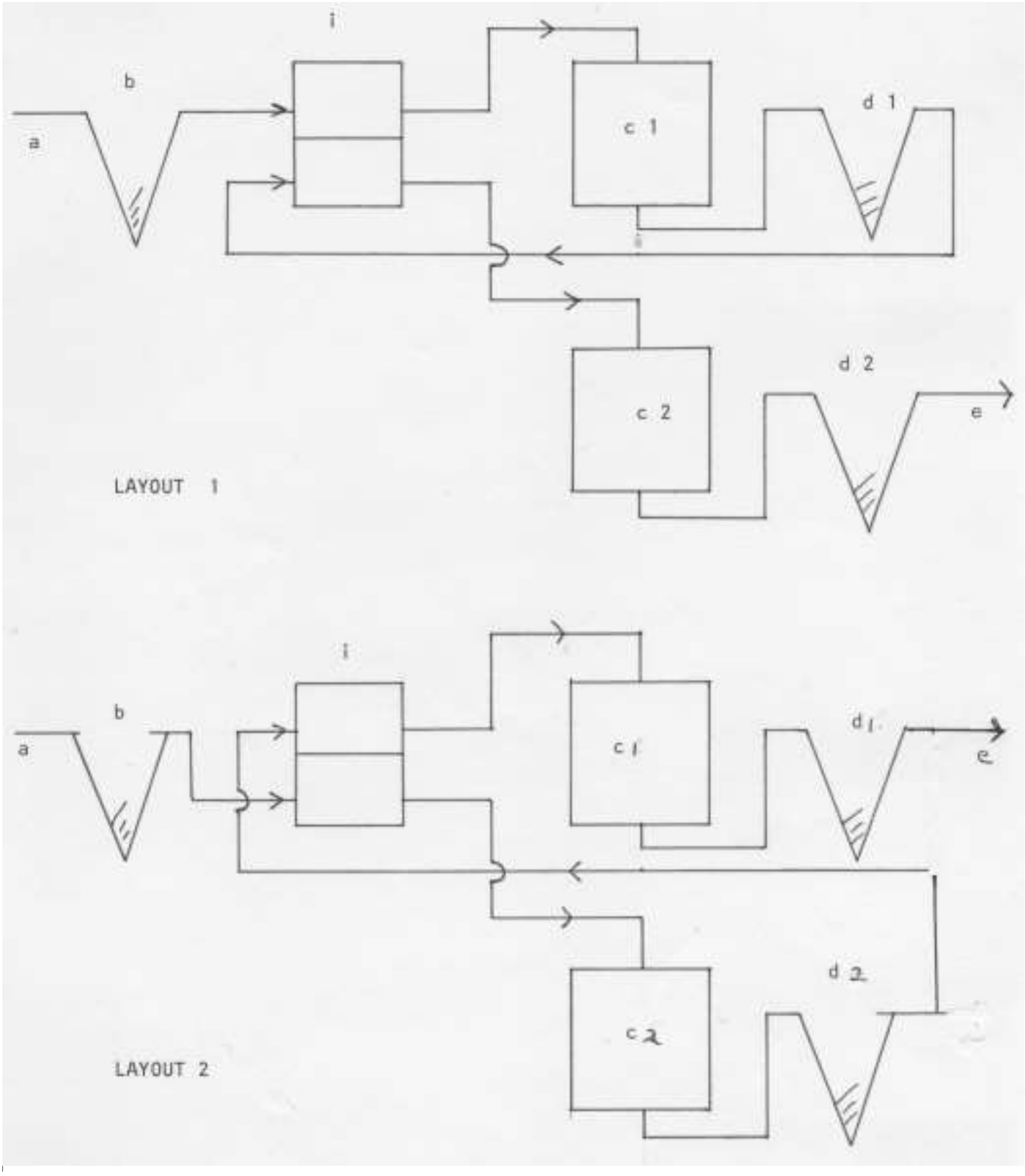


Figure 25 – FLOW PATHS FOR ALTERNATING SEQUENCE OPERATION OF BIOLOGICAL FILTERS.
(Sludge lines not shown – Symbols have same meaning as in Figure 24)

6.3 CLASSIFICATION OF BIOLOGICAL FILTERS ON THE BASIS OF LOADING RATE.

In section 6.2, biological filters were classified according to the layout of the filter or filters. They may also be classified according to the loading rate imposed on the filter. In increasing rate of loading these classifications are low rate, intermediate rate, high rate and super-rate. There are two ways of expressing the load on a biological filter:

1. in hydraulic terms;

2. in mass terms.

In hydraulic terms, the loading is measured in m^3 of wastewater fed per m^3 of media in the filter. Borrowed from the design of contact beds, (the forerunner of biological filter) the expression referred to as the surface loading of a biological filter is sometimes used though it is not suitable for low rate filters.

With contact beds, ventilation was available from the top only and so controlled the activity, so that the efficiency depended on the surface loading rate; but with the biological filter when ventilation is provided through the top and bottom openings, performance depends less on hydraulic load to surface ratio than on the loading rate per unit volume of filter medium. For high rate and very high rate filters, the surface loading rate becomes important because on it depends on the flushing effect characteristic of them.

In mass terms, the loading is measured in terms of oxidizable material fed per m^3 of media in the filter. Either the COD or the BOD may be used. In the USA, the BOD value is used whereas in South Africa the COD is usually used. Older books refer to the 4 hour Permanganate Value – PV_4. The mass loading may also be expressed in terms of population equivalents per m^3 of media. Here 110g COD or 55g BOD is taken to be one population equivalent.

6.3.1 Low Rate Biological Filtration.

The hydraulic loading rate is usually in the range 1 to 4 m^3/m^2-d. The organic loading rate is usually in the range 0.16 to 0.8kg COD/m^3-d. These filters are usually 1.8 to 3.0 m deep. Under the above conditions, these filters will usually achieve a high degree of nitrification i.e. the oxidation of ammonia to nitrate. The residual ammonia content of the effluent from a low rate filter is usually less than 10 mg/l as nitrogen when the effluent temperature is about 20°C. In conjunction with primary and secondary sedimentation, this system will usually achieve a BOD removal of 85%. Usually no recirculation is employed in low rate filtration.

6.3.2 Intermediate Rate Biological Filtration.

This filter is similar in design to the high rate filter and is often operated with recirculation. The hydraulic loading rate is usually in the range 4 - 10 m^3/m^2-d. The organic loading rate is usually in the range 0.7 to 1.5 kg COD/m^3-d. The major problem with the intermediate rate filter is that hydraulic loadings within this range apparently lead to clogging conditions. It may be necessary to use a large stone, up to 100mm, to reduce clogging. Nitrification is usually only partial.

6.3.3 High Rate Biological Filtration.

The hydraulic loading rate is usually in the range 10 - 30 m^3/m^2-d. The organic loading rate is usually in the range 1.5 to 3.0 kg COD/m^3-d. These filters are usually 1 - 2m deep. They are usually used with recirculation from 1 to 4 times the influent flow. Because of high loading and relatively shallow depth, sloughing is normally continuous and filter fly larvae are washed away. This eliminates problems with flies and clogging. Nitrification does not occur. In conjunction with primary and secondary sedimentation, this system will usually achieve a BOD removal of about 70%. Because of the high loading rate, the actual mass of COD removal is high.

In all three classifications above, the media consists, of stone, slag or similar material. In the low rate filters the media size is usually in the range of 25 to 75mm. In the high rate filter, media of up to 100mm may be used. This is to provide a larger passage for the water and sloughed off solids.

6.3.4 Super-Rate Biological Filtration.

Since the development of synthetic media with a large void space and a very high surface area to volume ratio: With stone, the surface area to volume ratio is about 40 m^2/m^3, whereas with synthetic media the ratio can exceed 150 m^2/m^3.

The hydraulic loading rate is usually in the range 30 to 60 m^3/m^2-d. The organic loading rate is usually in the range 3.0 to 6.0 kg/m^3-d. These filters are usually 3 to 12m deep. In conjunction with primary and secondary sedimentation, this system will usually achieve a BOD removal of about 50%.

6.4 DIFFICULTIES AND PROBLEMS OF BIOLOGICAL FILTRATION.

Most difficulties and problems in biological filtration are overcome by keeping the wastewater as fresh as possible and by not loading the filters above their design capacity.

These problems include:

1. ponding by clogging of the surface layers of the media;
2. development of bad odours;
3. the excessive breeding of Psychoda flies.

Problems may also occur when the distributing arms stand still during periods of no or very low flow. This can be overcome by recirculating secondary sedimentation tank effluent at low flow periods. Dosing syphons may also be used where a special tank fills up with wastewater and when full, discharges like a toilet cistern and turns the distributor arms for a short time, the tank then fills up and repeats the cycle. The use of recirculant is a more satisfactory method for operating during low flow periods.

The remedies for ponding on filters include:

1. Chlorination of the wastewater fed onto a filter;
2. the application of high pressure water jets onto the surface media;
3. the forking over of the top layers of the filter;
4. increasing the flow rate onto the filter by using a high recirculation rate;
5. taking the filter out of use for a few days and then dosing a high flow.

6.5 HUMUS OR SECONDARY SEDIMENTATION TANKS.

6.5.1 Introduction.

The solids that break off the media in the biological filter are washed out with the flow and need to be separated from the flow. This is done in the humus or secondary sedimentation tank. These tanks may be divided into radial flow tanks and upward flow tanks.

6.5.2 Mechanical sludge scraping and withdrawal in Radial Flow Tanks.

Two main types of scrapers are used:

1. Fixed bridge scrapers, where the bridge spans the tank and supports a walkway with hand railing providing access to the drive unit. The drive unit is mounted at the centre of the tank and consists of an electric motor driving a vertical shaft through reduction gearing. The shaft supports the two scraper arms and has a special watertight bearing at floor level.

The feed pipe is usually laid beneath the floor to the centre of the tank and then turns upwards to discharge into a deflector drum. The electricity supply cable for the motor is carried along the bridge and this arrangement is much simpler than the power supply to a rotating-bridge scraper. With tanks up to 15 m diameter, the bridge structure is usually fabricated from channels or joists (RSJ's) suitably braced, whilst the lattice design is used for tanks of larger diameter.

2. Rotating bridge scrapers, here there are two sub-types:

    i half bridge, extending over the radius of the tank,

    ii. full bridge, extending over the diameter of the tank.

Occasionally 3 arm bridges are used on very large tanks. A full-bridge scraper is usually centrally supported and one end of the bridge carries the drive motor, the other end being supported on an idler wheel. A half-bridge scraper is pivoted at the centre and either driven from the centre or by a motor on the outer end of the bridge.

The electrical supply system is more complicated than with a fixed bridge scraper, the cable having to the laid in a duct in or under the tank floor and carried up the side of the feed pipe or in the central column. With smaller tanks, the cable may be carried overhead, being supported by a catenary wire. When laid in a plastic duct, modern methods of sealing have reduced the earlier hazards. In either case, there is a slip-ring assembly at the tank centre, with a cable to the drive motor.

In the earlier design of scraper, a continuous helical blade was used but this was unable to automatically adjust itself to the profile of the floor if the screed finish was inaccurate. Blade arrangements now normally used in Humus tanks are either:

1. a series of blades arranged in echelon at an angle of about 45° to a rotating radial arm, the number depending on the diameter of the tank, See Figure 26 or;

2. a series of straight blade sections arranged to form a continuous volute. See figure 27.

With blades arranged in echelon the sweeping process is more gradual since, unless the blades overlap, the unit must make as many revolutions as there are blades to convey sludge from the periphery of the tank to the central hopper, although less load is imposed on the scraper mechanism. With the continuous volute, the sludge will be moved to the centre hopper much more quickly. This reduces the chances of gas production and the re-suspension of the sludge.

Scraper blades are often attached to hinged arms and are trailing, relying on their weight or on a spring-loaded action to prevent their being lifted from the floor by an accumulation of sludge. When on a hinged arm, the blade automatically adjusts itself to the profile of the tank floor.

Blades are edged with renewable rubber or rubber substitute wearing strips, some have a metallic edge. However, the weight of the trailing portion should not be such as to produce undue wear. Scraper blades are supported from a frame work carried from the bridge and if freely attached to the bridge structure, they can be lifted for examination without having to empty the tank.

On humus tanks, the peripheral speed of the scraper is usually in the range of 1.8 to 2.5 m/min.

An additional scraper is sometimes attached to the rotating framework carried by the bridge to maintain the cleanliness of the vertical side wall of the tank.

In the radial flow tank, sludge is withdrawn from the central hopper through a pipe running under the floor of the tank to an external inspection chamber. Here the rate of withdrawal can be controlled by a telescopic screw-down bell mouth weir or an adjustable straight weir.

In both horizontal flow and radial flow tanks, the sludge control valves may be hand operated, power operated by remote control or may even be partially or fully automatic.

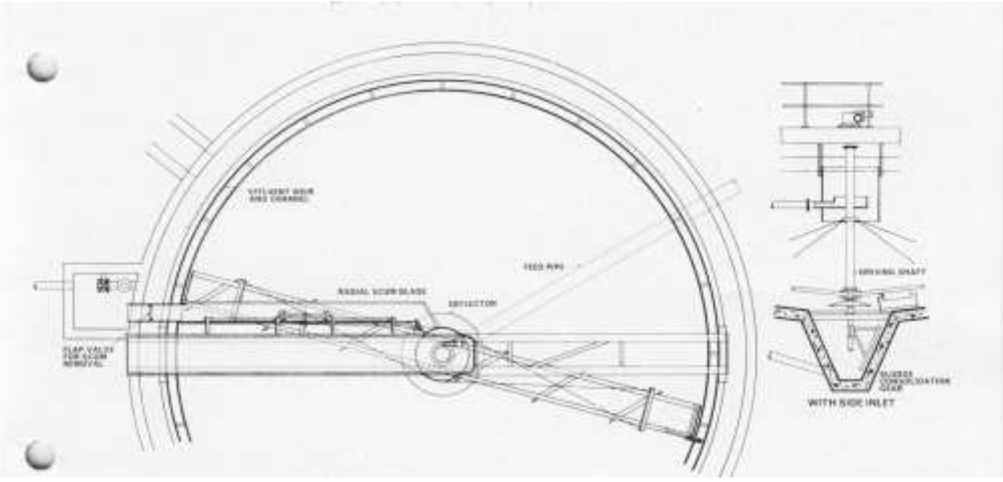


Figure 26 – SCRAPER BLADES IN ECHELON LAYOUT.

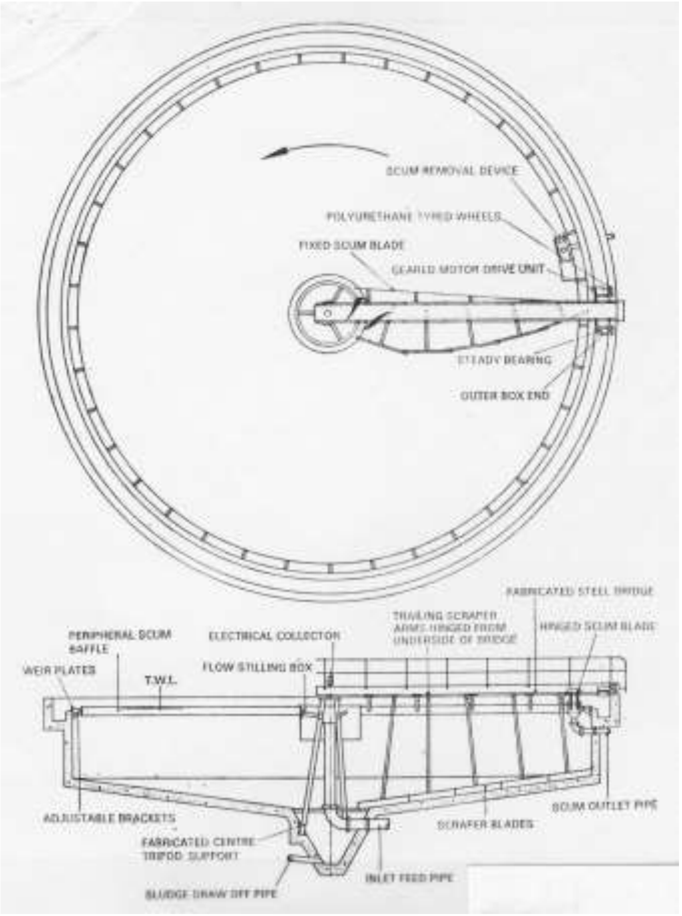
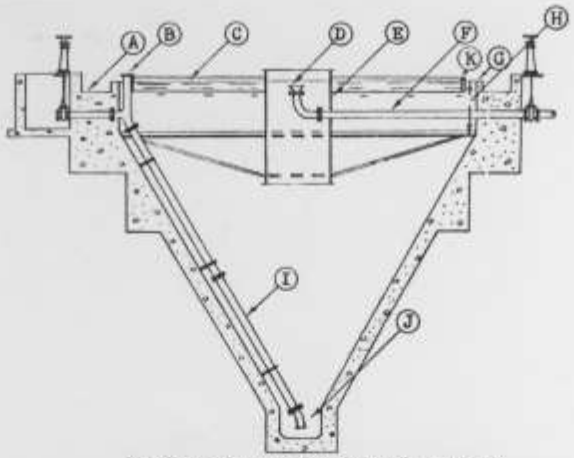


Figure 27 – SCRAPER BLADES IN CONTINUOUS VOLUTE LAYOUT.

6.5.2 Mechanical Sludge Scraping and Withdrawal in Vertical Flow Tanks.

It is usually not necessary to have scrapers in vertical flow tanks, since the sludge will slide down the steeply sloping sides, and will collect at the bottom. Sludge is withdrawn from the bottom through a pipe located inside the tank and terminating a short distance above water level. At least 1.2 m, and preferably 1.5 m below water level the pipe has a horizontal branch which discharges into an inspection chamber at the side of the tank. Sludge is withdrawn through this branch, whilst the vertical extension is used as a rodding "eye".

Sludge is withdrawn under pressure due to a differential head and discharges over a telescopic screw-down bell mouth weir or a swinging arm. The mouth of the sludge pipe should be 1.5 pipe diameters above the bottom of the tank to prevent blockage at this point, and if it is chambered so that it is in a vertical plane, this will facilitate both withdrawal of sludge and rodding, if this becomes necessary. The sludge pipe should be at least 150 mm diameter. See Figure 28.



Main features of an upward-flow primary sedimentation tank.

A, Effluent channel; B, rodding eye; C, top water level; D, bellmouth inlet; E, stilling box; F, feed pipe; G, peripheral weir; H, side water depth; I, sludge pipe; J, sludge sump; K, scum baffle.

Figure 28 – UPWARD FLOW HUMUS TANK.

The solid waste matter resulting from the purification processes in bio-logical filtration is known as humus and is discharged continuously in suspension in the effluent. The major portion of the humus is removable from the effluent by plain sedimentation in a humus or secondary sedimentation tank. These are usually similar in design to the primary sedimentation tanks. The humus sludge is drawn off either intermittently or continually from these tanks and is usually returned to the raw wastewater for removal in the primary sedimentation tank. A certain amount of the humus in the effluent is not removed by plain sedimentation and if a higher quality effluent is required then the humus tank effluent must undergo tertiary treatment to remove the non-settleable suspended solids.

The humus is flocculent, light and putrescible, it can be easily lifted into the effluent and more or less continuous removal is therefore desirable. This humus is similar in some ways to activated sludge and may be dealt with in much the same way, for example, by returning it to the primary sedimentation tanks in which it settles together with the primary sludge and from which the mixed sludge is removed for further treatment.

Sludge volumes, water contents and organic constituents are greater in high-rate operation than in low-rate operation. The solids content of low-rate humus sludge is usually 4 - 7% m/v and of high rate humus sludge usually 3 - 5% m/v.

Because the humus sludge is usually returned to the primary sedimentation tank for resettling with the primary sludge; it does not matter if moderate quantities of excess water are removed with the sludge from the humus tank.

6.6 LOADING RATES.

The typical retention time for a humus tank is 1.3 to 2.0 hours.

The typical surface loading rate is 35 - 40 m^3/m^2 - d.

WASTEWATER TREATMENT.

PART 7

ACTIVATED SLUDGE.

7.1 INTRODUCTION.

The biological filtration process of wastewater treatment may be termed a supported growth system, because the active zoogleal mass is supported on a medium such as stone. The activated sludge process on the other hand may be termed a suspended growth system, because the active zoogleal mass (called activated sludge) is suspended in an aqueous medium.

The activated sludge process of wastewater treatment had its origin in attempts to purify wastewater by blowing air into it. The process received its name from observations that:

1. flocs teeming with living organisms developed spontaneously during prolonged aeration of wastewater in a tank;
2. these flocs settled out when agitation and aeration of the tank contents were stopped;
3. addition of a new batch of wastewater to the growing volumes of settleable, activated sludge eventually produced a high degree of purification in a reasonable length of time.

Like other treatment methods, the activated sludge process was first operated on a batch wise basis, later continuous flow was found to be feasible.

As currently practiced, the process involves:

1. the continuous return of activated sludge to the aeration tank influent - known as returned activated sludge or RAS;
2. the wasting of sludge because the quantity of sludge is continuously increasing as new cells are synthesised, and in order to keep the total quantity of sludge in the system constant some sludge must be removed from the system periodically, usually daily. The sludge removed is called the excess, surplus or waste activated sludge - WAS;
3. the aeration and mixing of the sludge - wastewater mixture, called the mixed liquor, to keep it aerobic and to bring the sludge in contact with the influent suspended and dissolved materials;
4. the stirring of the mixed liquor in the anaerobic and anoxic zones to keep the sludge in suspension;
5. the sedimentation of the mixed liquor to separate the activated sludge.

The activated sludge process operates under water. Neither odour nor fly nuisance is attached to it and its performance is less affected by temperature than the biological filtration.

Activated sludge floc is formed in wastewater by the aerobic growth of bacteria. The zoogleal matrix synthesised by them generally includes protozoa and other living organisms, as well as organic debris from the wastewater. The various organisms in the flocs digest and utilize organic matter in the wastewater for energy and growth. Dissolved, colloidal and suspended organic matter is assimilated. The impurities that are converted into cell substance and matrix are thereby rendered settleable.

Activated sludge is similar in many respects to the biological filter film. However, it is given the advantage of mobility which makes for repeated contacts between the floc and the wastewater.

Because of this, the activated sludge process is a very effective form of biological flocculation.

In order to ensure that there is always sufficient sludge in contact with the wastewater in the aeration tank, activated sludge is returned from the underflow from the secondary sedimentation tank into the aeration tank at the point or points where the wastewater enters the aeration tank.
As the activated process is basically an aerobic process, it must be supplied with dissolved oxygen for a large percentage of the time. For fairly short periods, some of the zoogleal mass can utilize nitrates as an oxygen source but the organisms must then pass into zones where free dissolved oxygen is again readily available. This aspect is covered in greater detail in the section dealing with biological excess biological removal. The methods employed to introduce air (oxygen) into the mixed liquor will be covered later.

There are several layouts of works for treatment using the activated sludge process will be covered first. This followed by methods of aeration and the use of non-aerated zones.

7.2 CONVENTIONAL ACTIVATED SLUDGE PROCESS.

In this system, the wastewater undergoes preliminary treatment and primary treatment before passing into the aeration tank and then into the secondary sedimentation tank. The waste activated sludge is often returned to a point upstream of the primary sedimentation tank and is settled with the primary sludge to give a mixed sludge. Sludge production is about 0.5 kg sludge/kg BOD at 20 day sludge age. The hydraulic displacement through the aeration tank is usually plug flow.

The reactor is shown as a single tank. The various zones will be identified and discussed in later sections. The simple block diagram is shown as figure 29. The dashed line is an alternative sludge handling route via the Primary Sedimentation Tank. Sludge handling is covered in later sections.

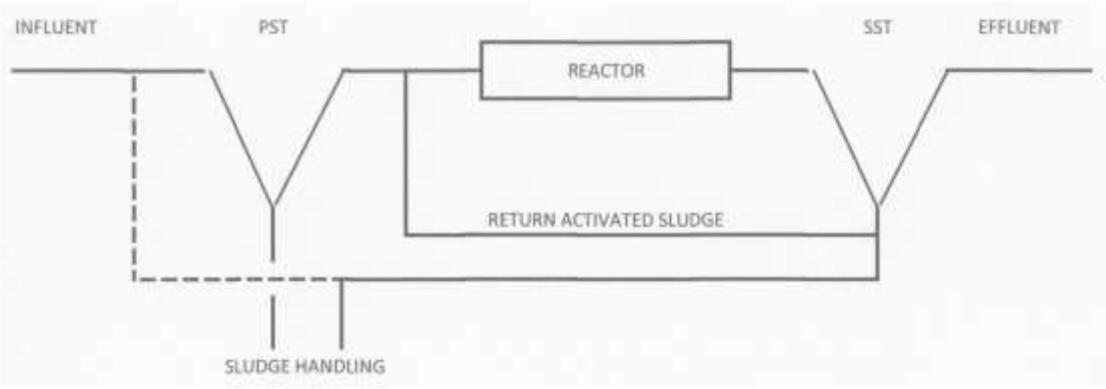


Figure 29 – SHOWING MAJOR COMPONENTS OF A CONVENTIONAL ACTIVATED SLUDGE WORKS.

7.3 EXTENDED AERATION ACTIVATED SLUDGE PROCESS.

With the conventional process, digesters or some other sludge treatment is needed before the mixed sludge can be dewatered and disposed of. This means that staff are needed to control the pumping of sludge to the digester, removing supernatant from the digesters and withdrawing sludge for dewatering.

In the extended aeration system, low F : M ratios are used; there is usually no primary treatment. The wastewater after preliminary treatment passes straight into the aeration tank. Aeration periods of 24 to 36 hours are used so that a large quantity of sludge may be kept in the system without having too high an MLSS.

Because of the long sludge age, often greater than 30 days, endogenous decomposition of the activated sludge takes place, resulting in a highly stabilised wasted sludge being produced that may be dewatered on drying beds without further treatment. The sludge production is less than when using the conventional process and may be as little as 0.3 kg sludge/kg BOD removed.

The reactor is shown as a single tank. The various zones will be identified and discussed in later sections. The simple block diagram is shown as figure 30. Sludge handling is covered in later.

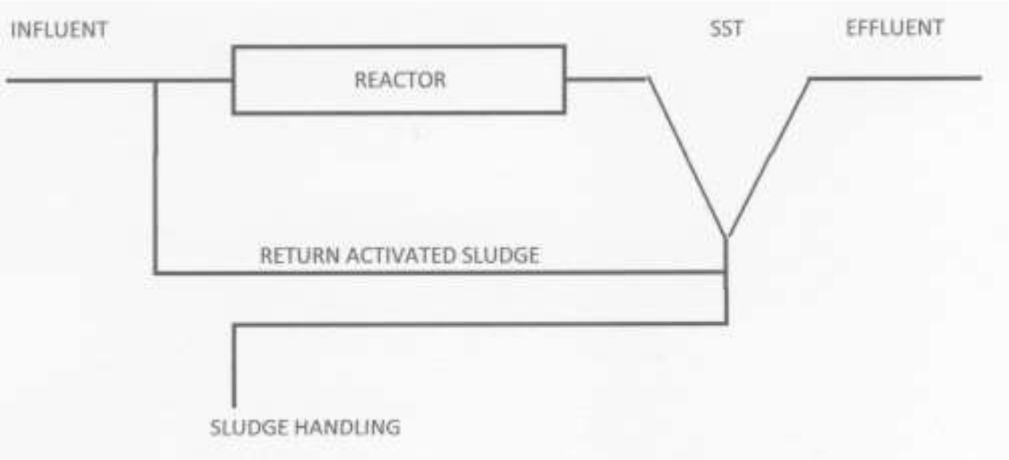


Figure 30 – SHOWING MAJOR COMPONENTS OF AN EXTENDED AERATION ACTIVATED SLUDGE WORKS.

Among the advantages of the extended aeration process are:

1. the works as a whole is much simpler;
2. there are no PST's or digesters to be built or to be operated;
3. there is only one type of sludge to deal with;
4. usually complete nitrification takes place.

Among the disadvantages:

1. more power is needed because more aeration is needed;
2. a larger aeration tank is needed;
3. a thin sludge is produced, that usually dewaters well only when placed in thin layers on the drying bed.

7.4 METHODS OF AERATION.

There are two main methods for aerating the mixed liquor:

1. Diffused air aeration;
2. mechanical or surface.

Both diffused air and mechanical units perform three essential functions:

1. they transfer the oxygen to the mixed liquor and keep the sludge aerobic;
2. they circulate the sludge in the aeration tank and create contact opportunity between the sludge and the wastewater;

3. they keep the sludge from settling to the tank bottom where it is ineffective and quickly becomes septic.

In diffused-air units, oxygen is transferred to the mixed liquor from compressed air bubbled up through the liquid. In mechanical aeration units, absorption of oxygen may be induced in either or both of two ways:

1. by rapidly circulating the mixed liquor past the water-air interface at the surface of the liquid;

2. by lifting the mixed liquor from the water surface and spraying it over the surface in droplets.

The diffused air method of aeration may be sub-divided into two groups:

1. fine bubble aeration;

2. coarse bubble aeration.

In fine bubble aeration, the bubble size is about 2 mm diameter. Because the bubbles are small, the efficiency of transfer of the oxygen from the air in the bubble to the mixed liquor is better than with larger bubbles. Oxygen transfer efficiencies of up to 25% may be obtained in a 4 metre water depth.

Figure 31 – FINE BUBBLE DIFFUSER FLOOR LAYOUT (Under construction)

In coarse bubble aeration, transfer efficiencies will be in the range of 5 - 10% depending on the bubble size. Diffusers are available in many shapes and sizes. Some are round and are fixed to the bottom of the tank, other are cylindrical and may be removed from the tank while the tank is still in operation, for cleaning.

Different diffuser layouts are used:

1. the diffusers may be placed along one wall only so that the mixed liquor is lifted upward on that side and a rotary flow is induced;

2. the diffusers may be placed down the centre of the tank so that the mixed liquor is lifted upward in the centre and two rotary flow patterns are induced;

3. the diffusers may be placed down two walls so that the mixed liquor is upwards at the sides of the tank and two rotary flow patterns are induced;

4. the diffusers may be laid en masse across the tank so that upward and downward currents are found all over the tank as per figure 31 above.

Surface or mechanical aerators may be subdivided into two groups:

1. vertical shaft aerators, such as Simcar, Simplex, K.S.B.;
2. horizontal shaft aerators, such as Kessener brush.

Figure 32 – EXAMPLE OF VERTICAL SHAFT SURFACE AERATOR.

Figure 33 – EXAMPLE OF HORIZONTAL SHAFT SURFACE AERATOR.

As the oxygen demand at the inlet end of the aeration tank is much greater than at the outlet end, high dissolved oxygen contents are often found at the outlet end while during high loads often the dissolved oxygen content at the inlet end dropped too low. To overcome this, virtually all works have tapered aeration, i.e. the rate of aeration is greater at the inlet end and decreases towards the outlet end. This can be done by using more diffusers at the inlet end and fewer at the outlet end as shown in figure 34. The dome density would be about 4.5 per m^2 at inlet reducing to about 2 per m^2 near the effluent weir. In the case with surface aerators, this could be by using more or larger surface aerators at the inlet end

Figure 34 – AN INDICATION OF THE RELATIVE DENSITY OF DIFFUSER DOMES OR OF SURFACE AERATORS

Another way to reduce the high oxygen demand at the inlet end is to use a completely mixed system. Here surface aerators must be used as by their rotation they move the mixed liquor in a horizontal direction as well as in a vertical direction. On entering the aeration tank, the wastewater is quickly dispersed throughout the tank. One disadvantage of this system is that some of the wastewater has a short retention time and if a very strong wastewater is being treated some ammonia may still be present in the aeration tank effluent. This may be overcome by splitting the aeration tank into two sections, a completely mixed system followed by a plug flow system as shown in figure 35.

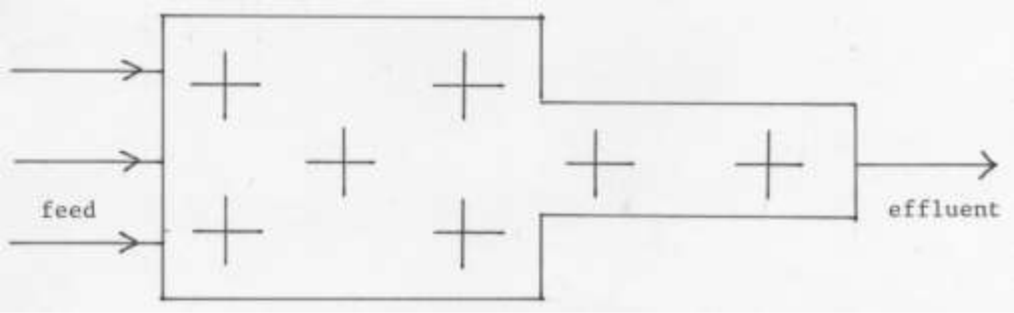


Figure 35 – COMPLETELY MIXED PORTION FOLLOWED BY PLUG FLOW PORTION.

It has been seen above that tapered aeration or the use of completely mixed systems have been used to try to match the high oxygen demand at the inlet end of the aeration tank. It has been seen that a completely mixed system can only be used with surface aerators unless some additional mixing or pumping system is included.

An older method, no longer used, is step loading. Here the wastewater enters the aeration tank at a few points. In this system, either diffused air or surface aerators may be used. With this system, the organic load is distributed over about 75% of the volume of the aeration tank; usually no wastewater is fed into the last 25 % of the aeration tank. This is to prevent partially treated wastewater leaving the aeration tank. This layout is not used where anoxic or anaerobic zones are used for nitrogen and phosphorus removal.

7.5 NITRIFICATION.

This has been defined as the oxidation of ammonia to nitrate.

This is actually a two-step reaction. In the first step, the ammonium ion is oxidised to the nitrite ion by the chemo-autotrophic bacteria Nitrosomonas. Chemo-autotrophic means that it gets its energy from the reaction of simple inorganic chemicals. The reaction is:

$$2NH_4^+ + 3O_2 \quad \rightarrow \quad 2NO_2 + 4H^+ + 2H_2O + 2e^-$$

The reason why the ammonium ion is shown in the reaction is because at pH7 - 8 in the range where the nitrifying bacteria are most active, most of the ammonia is present as the ammonium ion. It is important to note that hydrogen ions are produced. If the buffering capacity of the water is low then the pH of the mixed liquor may drop and unless the bacteria are acclimatised, this stage of nitrification may cease.

The second step of the nitrification process is the oxidation of nitrite to nitrate by the also chemo-autotrophic Nitrobacter bacteria. The reaction is:

$$2\ NO_2^- + O_2 \quad \rightarrow \quad 2NO_3^-$$

The overall reaction is thus:

$$4NH_4^+ + 4O_2 \rightarrow 2NO_3^- + 4H^+ + 2H_2O + 2e^-$$

1 mg N needs 4.57 mg Oxygen for conversion from the ammonium ion to the nitrate ion. Usually the dissolved oxygen needs to be at about 2 mg/L for complete nitrification.

One of the main problems with not nitrifying is that the ammonia exerts a high oxygen demand in the receiving water and that the adverse reaction of fish to ammonia at low dissolved oxygen contents.

The modern approach to the problem of not wanting ammonia or nitrates in the treated effluents, is to nitrify fully and then to denitrify as fully as possible. Denitrification is covered in the next section.

The micro-organisms responsible for the process of nitrification are not tolerant to sudden pH changes and are easily inhibited by toxic or harmful chemicals entering the system. This is why when a treatment works is under stress, it is the nitrification that stops first.

7.6 DENITRIFICATION.

In this process; nitrates are utilised by facultative aerobic bacteria as an oxygen source in the absence of dissolved oxygen. In this case, nitrates are reduced to nitrogen gas and this escapes from the system. Among the bacteria that will denitrify are: Pseudomonas, Micrococcus, Achromobacter and Bacillus species

In addition to having nitrates present but dissolved oxygen absent, there must be sufficient food for the facultative aerobes to metabolise. This food may be in a stored form absorbed to the sludge. The condition where nitrates are present but dissolved oxygen is absent is called anoxic.

The food may be present as the organics present in the wastewater or in the respiration of the activated sludge or it may be added usually in the form of methanol. Molasses has been tried but is not very successful.

The flow sheet of a typical 4 stage Nitrification - Denitrification works using raw wastewater as the electron donor is shown in figure 36.

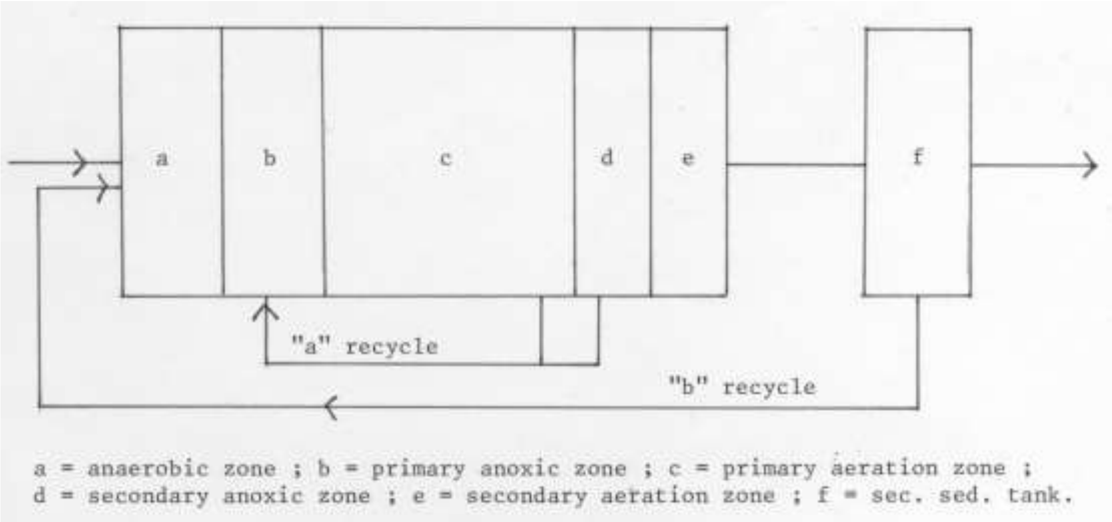


Figure 36 – LAYOUT OF THE 4 STAGE ACTIVATED SLUDGE TREATMENT WORKS.

The "b" recycle is the usual recycle to return the sludge from the S.S.T. The "a" recycle is the high nitrate recycle. The nitrate content of the "b" recycle will be low.

From this process, the total nitrogen removals of up to 90 % may be obtained with effluent ammonia and nitrate contents of less than 5 mg/l each.

It is often found with this system that enhanced phosphorus removals by the sludge occur. The waste activated sludge must not be wasted via the PST'S but must be thickened and dewatered under aerobic conditions. Under anaerobic conditions, phosphates are released from the sludge into the water phase.

2020 NOTE – THE SUBJECT OF BIOLOGICAL EXCESS PHOSPHORUS REMOVAL WILL BE COVERED IN VOLUME 4 IN THIS SERIES OF PROCESS CONTROLLER GUIDES.

In nitrification, about 6 mg/l of alkalinity is removed for each mg/l of ammonia ion as nitrogen oxidized. In denitrification, about 3 mg/l of alkalinity is recovered for each mg/l of nitrate ion as nitrogen reduced. Using methanol as a carbon source, the denitrification reaction is:

$$6NO_3^- + 5CH_3OH \rightarrow 3N_2 + 5CO_2 + 7H_2O + 6OH^-$$

7.7 THE SLUDGE AGE (SRT – SOLIDS RETENTION TIME).

The floc that develops in the aeration tank may be likened to the bio-mass that develops on the media in a bio-filter; the wasting of the activated sludge may be likened to the unloading of the biomass from the filter. The one major difference between the bio-filter and the activated sludge methods of treatment is that the quantity of bio-mass in the aeration tank may be varied by the Process Controller. Another important difference is that the dissolved oxygen content in the aeration tank may also be varied by the Process Controller.

By varying the quantity of sludge in the system, the ratio between the organic load entering the aeration tank and the sludge mass may be altered. This ratio is called the F: M ratio, the Food : Microorganisms ratio. Usually the quantity of food is measured as kg BOD per day entering the aeration tank and the quantity of micro-organisms is measured as kg MLSS (mixed liquor suspended solids), in the aeration tank. The F: M ratio may also be called the sludge loading rate.

As bio-mass is continuously being produced and then wasted, the concept of sludge age may be introduced. This may be defined as the mean residence time of the sludge in the system. This sludge age may be determined in many ways: - some of those more commonly used are:

1. where mixed liquor is wasted, the easiest calculation using volumes is:

$$\text{Sludge age} = \frac{\text{Volume of reactor}}{\text{Volume of mixed liquor wasted per day}}$$

2. where return sludge is wasted then it is necessary to determine the MASS wasted per day:

$$\text{Sludge age} = \frac{\text{Total mass of suspended solids in the reactor}}{\text{Total mass of sludge wasted per day}}$$

In these calculations, the quantity of sludge in the secondary sedimentation tank is ignored as the quantity should be very much less than the quantity of sludge in the aeration tank.

For a given organic and hydraulic loading and a given aeration tank volume, increasing the sludge age by increasing the mass of sludge in the system, lowers the F : M ratio and vice-versa. The activated sludge process may be broadly separated into 3 mains group characterized by the F: M ratio or by the sludge age.

1. the high rate system having a sludge age of 1 - 5 days and an F : M ratio of up to 0.8;
2. the conventional system having a sludge age of 10 - 20 days and an F : M ratio of 0.15 - 0.2;

3. the low rate of extended aeration system having a sludge age of greater than 30 days and an F : M ratio of 0.1 or less.

Except in extended aeration and aerobic sludge digestion systems, only a minor portion of the polluting organic matter absorbed by the activated sludge is actually decomposed, the major portions forms additional sludge and must be disposed of regularly (usually daily) to prevent its accumulation in the system.

The longer the sludge age, the larger the proportion of the organic load (measured as BOD) that is actually satisfied. The following table gives an indication of the percentage actually satisfied:

TABLE 5 – INDICATING ORGANIC LOAD SATISFIED vs SLUDGE AGE.

SLUDGE AGE – d	**% BOD satisfied**
3	12
7	25
20	50
40	70

It is seen that except at long sludge ages, little of the influent organic load is actually satisfied, what the activated sludge and also the biological filter process is doing is making the fine colloidal and dissolved organic impurities settleable so that they can be removed for further treatment.

The waste activated sludge must be further treated by one of the following methods:

1. drying on drying beds with or without intermediate thickening;
2. thickened and passed into an anaerobic digester;
3. passed to a point upstream of the primary settling tanks and allowed to settle out with the primary sludge. This sludge is called - Mixed Primary/Waste Activated.

The age of the sludge is a measure of its putrescibility. Usually the minimum age at which sludge may be dried on drying beds without causing a nuisance is about 20 days. It is usual to operate extended aeration works at about 25 days sludge age.

It is usual to waste activated sludge from the secondary sedimentation tank underflow i.e. it is the same liquor that makes up the sludge return. Using this method, it is not always possible to regulate the sludge age very accurately. For the most accurate control of sludge age, it is necessary to waste mixed liquor directly from the aeration tank, eg. for a 25 day sludge age one would waste 1/25 of the volume of the aeration tank every day.

7.8 SECONDARY SEDIMENTATION TANKS.

7.8.1 Introduction.

In order to recover the activated sludge leaving the aeration tank, the mixed liquor is passed into a secondary sedimentation tank.

The sedimentation operations may be classified approximately as falling into 4 separate categories. These are:

1. Class 1;
2. Class 2;
3. Zone Settling;
4. Compression Zone.

Classes 1 and 2 relate to low concentrations of suspended solids and are dealt with below. Zone and compression sedimentation relate to higher concentrations of suspended solids and are dealt with later under sludge handling. The various classes are shown in figure 37.

The class 1 sedimentation referred to above relates to discrete particles and class 2 to flocculent particles. In the activated sludge process, one is working with flocculent particles.

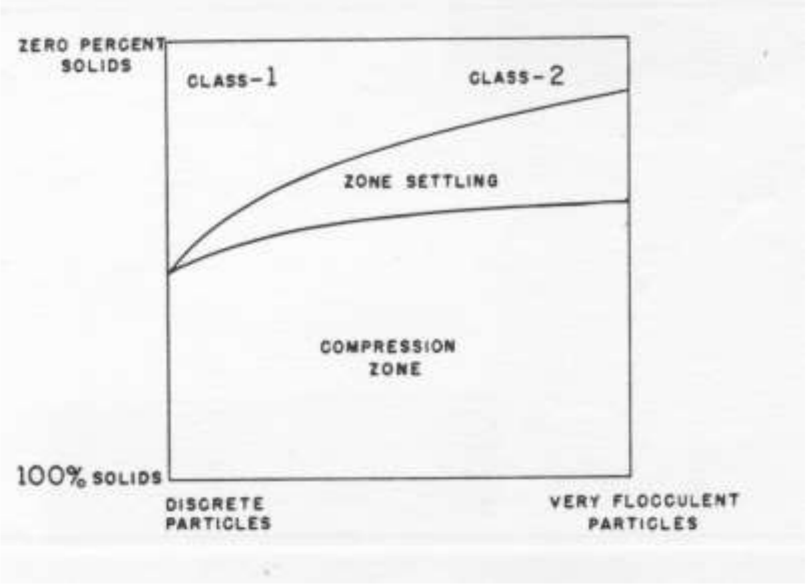


Figure 37 – CLASSIFICATION OF SEDIMENTATION PROPERTIES

7.8.2 The settling characteristics of flocculating particles.

The settling properties of suspensions of flocculating particles differ from those of non-flocculating particles in that the flocculating properties of the suspension must be considered along with the settling characteristics of the particles. In this case, the heavier particles having a greater settling velocity overtake and coalesce with the smaller and lighter particles to form even larger particles with an increased rate of settling. The opportunity for particle-particle contact increases as the depth of the sedimentation tank increases. As a result, the removal of suspended matter depends not only on the settling rate but on depth as well.

7.9 FACTORS AFFECTING SEDIMENTATION.

Among the factors affecting sedimentation are:

7.9.1 The size, shape and relative density of the floc.

The larger the floc for a given shape and relative density; the higher the sedimentation rate. The more spherical the shape, the less the drag as the particle passes through the water and hence, all other factors being equal, the greater the terminal settling velocity. The higher the relative density of the particle; the higher the sedimentation rate. This is the reason why a dense floc albeit of smaller particle size is preferred. A large feathery floc (such as a bulking sludge) will have a relative density closer to that of the water and will also have a poor shape factor with a high drag coefficient.

7.9.2 The surface loading rate.

The surface area loading rate of a sedimentation tank is the most important in determining the efficiency of the removal of suspended solids. The surface area loading is usually expressed as $m^3/m^{2\cdot}h$, it is seen that this is dimensionally equivalent to m/h, which is a velocity. The surface loading rate may thus be equalled to the effective upward velocity of the water. If the upward velocity is too high and exceeds the settling velocity of the sludge; then solids will be lost out of the tank.

The upwards velocity should not exceed 1 m/h at maximum wet weather flow or about 0.5 m/h at average dry weather flow.

7.9.3 The average retention period for sedimentation.

This is calculated by dividing the volume of the tank (usually in m^3), by the flow rate (usually in m^3/h), to give the retention period in hours. However, some of the water will pass through the tank in less than this theoretical retention period while some of the water will take more than this time. If the tank is of poor design, short circuiting may take place, which the result that some of the water may have a very short retention period and this can result in suspended solids being carried over the overflow weir. Based on raw wastewater flow, the dry weather retention periods is usually about 4 to 8 hours.

7.9.4 The weir overflow rate.

If the weir overflow rate (usually measured in m^3/m-d, i.e. m^3 of flow per day per metre of weir length) is too high, then excessive upward velocities can occur near the overflow weir which, if they exceed the sedimentation rate of the floc, will draw the sludge particles up and over the weir.

The weir overflow rate should not exceed 100 m^3/m-d (4 m^3/m-h).

Because of the great importance of the returned sludge being in a good condition in the activated sludge process, the proper design and operation of the secondary sedimentation tank in the works is vital. Because the process is aerobic, except the anoxic zones needed for denitrification; the sludge that settles in the S.S.T. must be removed as soon as possible, and returned immediately to the aeration tank. Because of this need to return the sludge quickly and because of the light flocculent nature of the sludge, it is usually removed from that area of the floor upon which it falls either by suction or by air lift. In radial flow tanks, the floors are usually flat. The layout will be similar to that shown in figure 38.

As with the other sedimentation tanks described before, both fixed and rotating bridges are used.

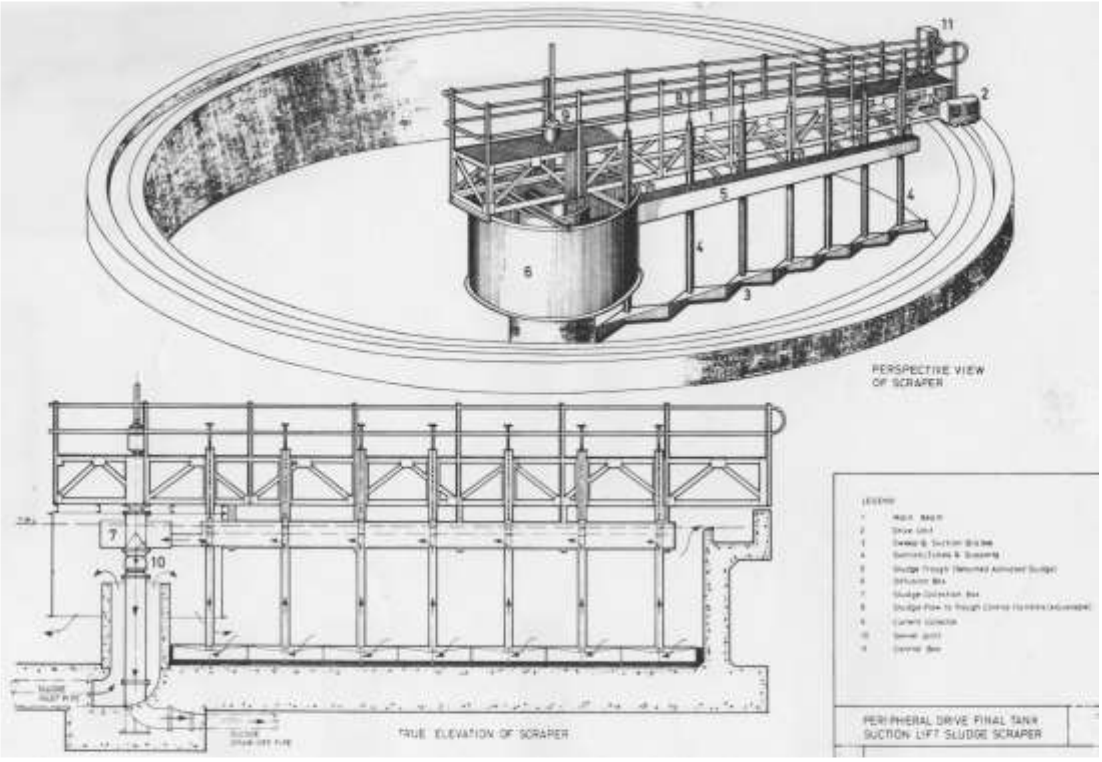


Figure 38 – SLUDGE REMOVAL BY SYPHON OR AIR LIFT.

7.10 THE PURE OXYGEN PROCESS.

In general, the limiting factor in the loading of an activated sludge works is the rate of transfer of oxygen from the gas phase to the liquid phase. Increasing the power input by using more powerful aerators or blowers is not the answer, because the high energy input tends to damage the settling characteristics of the sludge.

The use of high purity oxygen (90 %) enables high dissolved oxygen levels to be maintained with low energy input to the mixed liquor. This contributes to a highly aerobic biological mass that flocculates and settles well. Because the sludge settles better and the rate of oxygenation is faster the M or microorganism content can be increased and so the F or food (COD or BOD load) can be increased. This means that for a given load, a smaller aeration tank may be used. This will save on the civil costs. Whether this system is more economical than the air activated sludge depends on many factors.

7.11. MONITORING AN ACTIVATED SLUDGE WORKS.

The tests dealt with in this section are only those that relate to an activated sludge works only. The general tests will be covered later. The parameters in an activated sludge works that must be measured are:

7.11.1 The Dissolved Oxygen Content in the Aeration Tank.

As the activated sludge process is an aerobic system, it is vital that the dissolved oxygen content be above the minimum for the process to function properly. There is some disagreement as to the minimum values but they may be taken 2 mg/L where virtually complete nitrification is wanted.

Having the D.O. content too high costs money as the efficiency of aeration decreases as the D.O. content of the liquid being aerated, increases i.e. as the D.O. deficit is reduced. Over-aerating sludge for extended periods can result in a poorly settling sludge developing.

Under-aerating for extended periods will resulting in the sludge becoming septic and starting to smell, poor COD removals will result and the sludge will have a poor settling characteristic. Nitrification will stop.

If the works is designed for biological denitrification, then it is essential that the organisms have nitrate as an oxygen source in the anoxic zones, and that the D.O. be sufficient in the aeration zones.

Ideally D.O. should be monitored continuously, but reading as often as possible to establish trends is the next best. The modern method to measure dissolved oxygen is by using a D.O. meter. These must be standardized before use. The method depends on the instrument. Chemical methods are not really suitable because of interaction of the sludge and certain of the chemicals used in the test.

7.11.2 The Settleable Solids in the Aeration Tank.

This is useful test that should be done every shift. An experienced Process Controller can tell much about a works just looking at the sludge. In this test, a grab sample of Aeration Tank Effluent is taken and poured into a glass one litre measuring cylinder up to the mark. The sludge is allowed to settle for 30 mins and the volume occupied by the sludge is then read and recorded. It is seen that this is a "volume in volume" determination. It gives no indication of the mass of the sludge in the system; this is determined below in section 15.6.2.

This test can be extended to be used on the sludge return as well; it gives warning of impending trouble. If the settleable solids of the returned sludge suddenly drops; then a syphon could have broken or blocked up.

Monitoring the settleable solids of the Aeration Tank Effluent, can give warning as to problems, if the settleable solids suddenly increases then a bulking or poor settling problem could be starting to occur. In this case, the test below should be done as soon as possible and repeated daily while the problem occurs.

7.11.3 The Mixed Liquor Suspended Solids.

This is a mass in volume measure of the quantity of activated sludge in the system. It is not an absolute measure because it does not differentiate between healthy and dead sludge nor between the vital organisms and organic debris. The MLSS is measured in mg/L and is usually determined by drying 100 mL of Aeration Tank Effluent at 105º C and weighing the residue.

A filtered sample must be dried similarly to correct for the dissolved solids content.

This test should be done at least once per week, and preferably more often. At the same time, it is advantageous to do a similar determination on the wasted sludge and if the volume of sludge wasted is known then the mass of sludge wasted may be determined.

The results obtained from 15.6.1 and 15.6.2 may be combined to give a parameter known as the Sludge Volume Index – SVI.

This is defined to be:

The volume in mL occupied by 1 g of sludge (dry basis) after 30 mins of settling.

It is calculated as follows:

A good sludge will have an SVI of 50 - 125 mL/g (note units) A poorly settling or bulking sludge may have a value of 250 or more.

The SVI test must not be regarded as the final say in the settleability of a sludge.

For example, for a MLSS of 4 000 mg/L, the two sludge settling curves show the same SVI of 100 mL/g. It can be seen that sludge "B" settles and compacts better than sludge "A". See figure 35.

Other variations of this test have been suggested. One includes reading the volume at 15, 30 and 60 mins, plotting the points and comparing values from day to day, as its settling velocity must be more than the upward velocity of the secondary sedimentation tank.

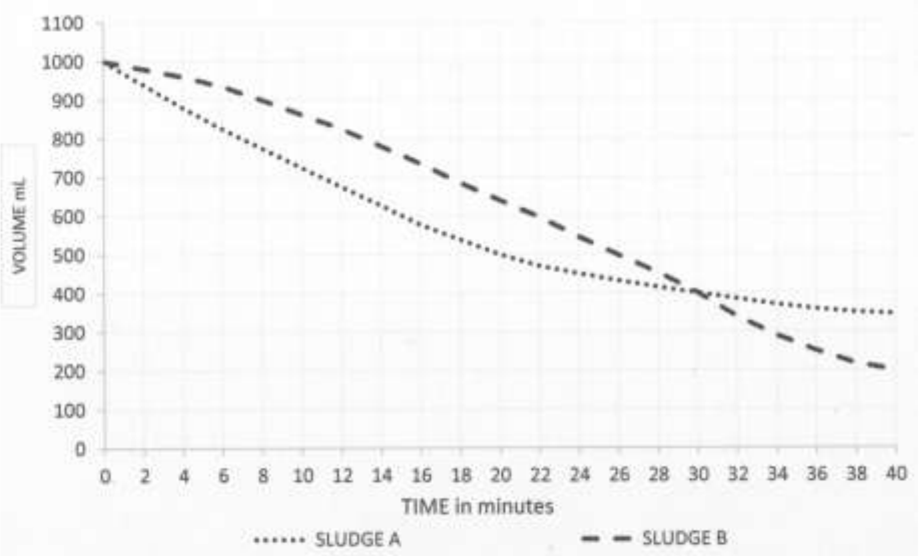


Figure 39 – SHOWING SETTLING CHARACTERISTICS OF TWO SLUDGES WITH SAME SVI.

7.12. SLUDGE BULKING.

Activated sludge may be overloaded if the influent wastewater is abnormally strong or the time of aeration is abnormally short. Deteriorating sludge is characterised by the following physical characteristics:

1. a stale odour;

2. poor colour often greyish or light in colour instead of a rich chocolate brown;

3. poor separation from the mixed liquor;

4. a tendency to rise in the secondary sedimentation tank, as its settling velocity may be less than the upward velocity of the secondary sedimentation tank;

5. the presence of few and relatively sluggish organisms.

Sludge that has deteriorated badly has a foul odour, settles very poorly in the settleable solids test, maybe greyish in colour, and is more or less lifeless. Poisoning of the sludge by shocks of toxic industrial wastes, including heavy metals, acids, phenols and mineral oils etc., has a similar effect.

When an activated sludge works has been poisoned one usually notices an increase in dissolved oxygen content. This is because the organisms are not able to use the oxygen. Usually Process Controllers make the mistake of reducing the amount of aeration as they think that they are just over-aerating, whereas in fact they should be INCREASING the rate of aeration.

Any sudden increase in dissolved oxygen content should be followed up by doing a sludge oxygen uptake determination in the laboratory. If the uptake rate has suddenly deteriorated then this indicates that the sludge is under stress and one must increase the dissolved oxygen content. If the works has been badly poisoned it may be necessary to increase the dissolved oxygen content to as much as 5 mg/l.

The oxygen uptake test will be described later in section 15.6.10.

Sludge bulking is the swelling of the sludge volume. True bulking, which may happen quite suddenly, is characterised by an increase in the SVI.

It is often first noticed by a gradual or sudden increase in the volume recorded in the 30 mins. settling test. The sludge may remain flocculated or a slimy deflocculated sludge may be formed.

The latter usually occurs as the result of the growth of filamentous organisms such as Sphaerotilus Natans.

The essential correctives for sludge bulking appear to include:

1. wasting more sludge in order to remove it from the system more rapidly, then hopefully when wasting is stopped, the fresh sludge produced will be of high quality;

2. increasing the dissolved oxygen content until the system corrects itself;

3. reducing the load on the works if possible, if PST's are used, then chemical treatment can be introduced in order to improve load removal in the PST's.

5. the addition of flocculating chemicals such as ferric chloride to the aeration tank.

7.13. ROTATING BIOLOGICAL CONTACTOR. (ROTATING DISC UNIT).

This process may be thought of in some aspects as a cross between biological filtration and activated sludge. It has the characteristics of the former in that the bio-mass grows on a fixed surface, although the surface happens to be moving slowly. It also has good resistance to shock loads, both organic and hydraulic. It has the characteristics of activated sludge in that a tank containing wastewater is used and that there is a mechanical agitation of the water to prevent settlement of organic matter.

A Typical layout is shown in figure 40.

The incoming wastewater will be treated in a septic tank so that solids in the influent are allowed to settle and be decomposed by anaerobic digestion. The effluent will contain fine solids that will be treated in the rotating biological contactor.

Basically the secondary process consists of a number of plastic discs mounted on a horizontal shaft, and placed in a tank with a contoured bottom. The discs rotate slowly in the wastewater with about 40 % of the surface area submerged. During rotation, the discs pick up a thin layer of wastewater, which flows over the disc surface and absorbs oxygen from the air. The fixed bio-mass film on the disc surface utilizes the dissolved oxygen in order to metabolise the organic material in the wastewater.

The disc unit will be covered over to keep the discs (usually made of plastic) out of the sun.

Excess biomass and other solids are continuously removed from the film fixed to the discs, by the shearing forces created by the rotation of the discs in the wastewater. This rotation also causes mixing, which keeps the sloughed solids in suspension so that they can be carried through each step to the secondary sedimentation tank.

The sludge produced by this process is similar to the sludge from a biological filter. The secondary sedimentation tank is desludged every few hours and the sludge pumped to the septic tank for digestion together with the influent wastewater. The septic tank will need to be desludged once every 6 months or so, depending on the capacity of the tank and the influent load.

This process is usually used only for small plants up to about 1 000 m³/d.

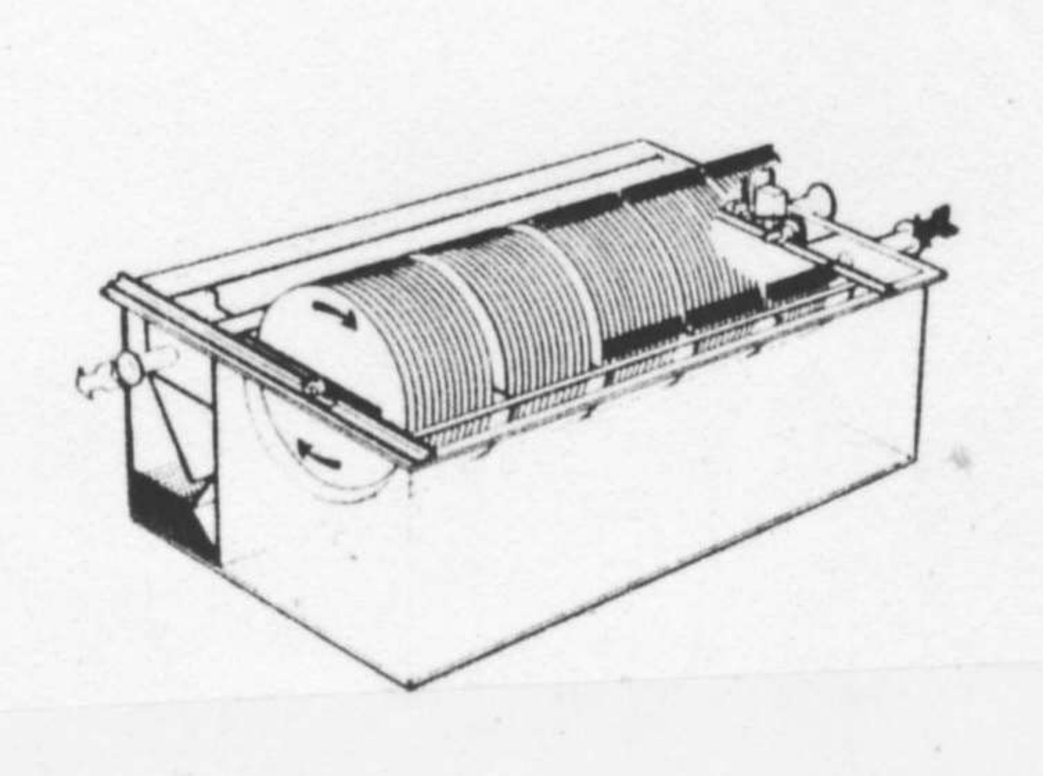


Figure 40 – LAYOUT OF A TYPICAL ROTATING BIOLOGICAL CONTACTOR.

7.14. COMPARISON OF SECONDARY TREATMENT PROCESSES.

Rotating biological contactors are not included in the comparisons.

TABLE 6 – COMPARISON OF BIOLOGICAL AND ACTIVATED SLUDGE TREATMENT PROCESSES.

	BIOLOGICAL FILTERS	ACTIVATED SLUDGE
1	Must have PST's, which means a primary sludge to be handled.	Extended aeration plants do not have PST's, but conventional plants do. Extended aeration plants have only waste activated sludge to be handled.
2	More head loss through the works especially for 2 stage plants.	Less head loss through the works.
3	Occupies more space.	Occupies less space.
4	Can handle shock loads more easily.	More susceptible to shock loads.
5	For complete nitrification, need very low rate filtration rate.	Will nitrify at quite high loading rates provided enough aeration available.
6	Not possible to get a high degree of denitrification without additional tanks and methanol addition.	Can get a very high degree of denitrification with suitable design.
7	Needs less skilled Process Control.	Needs much more skilled Process Control.
8	Generally more expensive to build but cheaper to operate.	Generally cheaper to build but more expensive to operate.
9	Not possible to get Biological Excess Phosphorus Removal.	With suitable design can get very good Biological Excess Phosphorus Removal.
10	Generally there is a Psychoda fly nuisance.	No fly nuisance.
11	Effluent quality reasonable. Generally not clear without further treatment.	A very high quality effluent with very low suspended solids is possible.

WASTEWATER TREATMENT.

PART 8

DISINFECTION.

8.1 THE NEED TO DISINFECT TREATED EFFLUENT.

Raw wastewater contains high numbers of a very wide variety of organisms both pathogenic (disease causing) and non-pathogenic. During primary treatment some organisms die or are removed from the water with the primary sludge.

The majority are removed during the secondary treatment stage.

The organism Escherichia Coli Type 1 has been used as an indicator organism. This organism is normally non-pathogenic and is found the bowel of humans, animals, birds, etc. It has been found that the rate of destruction or death of the E.Coli is similar to that of most pathogenic intestinal bacteria. The E.Coli content of raw wastewater is about 10^{12} per 100 mL, while the effluent from the secondary sedimentation tank is usually in the range 10^4 to 10^6 per 100 mL.

The General Standard to which most treatment works effluents must comply specifies an E. Coli content of nil per 100 mL, in many cases however relaxation to 1 000 per 100 mL has been granted.

It may be seen from above that it is unlikely that the effluent from the secondary treatment stage will comply with the standard. In many cases, tertiary treatment may be used to improve the quality of the effluent.

Over the years, there have been numerous instances of outbreaks of waterborne diseases. In virtually all cases these can be traced back in to unsatisfactory disposal of domestic and industrial wastewaters. In order to protect the receiving waters into which treated effluents are discharged, it is necessary to reduce the quantity of pathogenic and intestinal organisms discharged into the water. The three types of organisms that one is most concerned about are bacteria, viruses and amoebic cysts.

8.2 PRINCIPLES OF CHLORINATION.

The disinfection of wastewater effluent is usually accomplished by the use of gaseous chlorine, or chlorine compounds, because of the limitation of other procedures.

The object of chlorination of treated wastewater effluents is the destruction of bacteria through the germicidal effects of chlorine.

The problem of effective chlorination is to ensure:

1. uniform application of chlorine to all portions of the effluent being treated;

2. uninterrupted application of chlorine;

3. the selection of the dose of chlorine to meet the current needs of the specific effluent being treated.

Treated wastewater effluent consists of a complex solution of many substances, some of which influence chlorination amongst these are:

1. suspended solids may shield bacteria from the action of the chlorine;

2. residual organic matter may react with chlorine, so that it has only weak disinfecting

properties;

3. residual ammonia reacts with chlorine to form chloramines or combined residual chlorine which have much lower disinfecting properties than free residual chlorine;

4. treated effluent having low alkalinity and pH values of less than 7.2 is more easily disinfected than water with a high pH value;

5. nitrites react with chlorine and may also produce a false colour with the reagent normally used to detect residual chlorine namely ortho-tolidine. The ortho-tolidine-arsenite test may be used overcome this interference;

The rapidity of disinfection with chlorine is proportional to the temperature of the water, but at lower temperatures chlorine is more stable and will remain in the effluent for a longer period. The two effects compensate to some degree.

The period available for the interaction between chlorine and the constituents of the effluent is most important. The absolute minimum period should be 15 minutes.

8.3 THE REACTION OF CHLORINE WITH VARIOUS CHEMICALS IN THE WATER.

Chlorine is a powerful oxidizing agent and as such will oxidize a wide variety of both inorganic and organic substances in water. The degree of oxidation will depend on many factors such as temperature, concentration of chlorine, contact period etc.

The form in which chloride exists in water is dependent on the pH of the water. This is best illustrated in the figure below – figure 41.

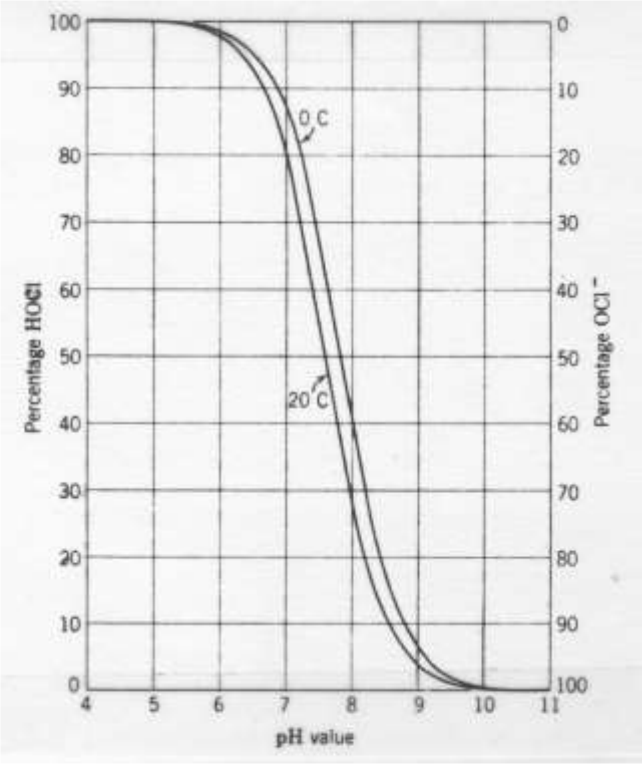


Figure 41 – CHLORINE SPECIES AS A FUNCTION OF pH VALUE.

When chlorine dissolves in water it forms hypochlorous acid (or hypo-chlorite ion) depending on the pH of the solution and hydrochloric acid:

The reactions are:

1. $Cl_2 + H_2O \rightarrow Cl_2$ aqueous
2. $Cl_2 + H_2O \rightarrow HOCl + HCl$
3. $Cl_2 + H_2O \rightarrow H^+ + OCl^- + HCl$

In the pH range usually found in treated wastewater effluent, the free-residual chlorine is present as a mixture of hypochlorous acid and hypochlorous ion

The hypochlorous acid is unstable and breaks up to yield hydrochloric acid and oxygen. It is this oxygen that is responsible for most oxidation reactions. In the disinfection process, it is the hypochlorous acid that is the agent that kills the bacteria. The hypochlorous ion is less effective.

Chlorine reacts with ammonia and ammonium compounds in water to form firstly monochloramines, then dichloramines and finally trichloramine or nitrogen trichloride.

$NH_3 + HOCl \rightarrow NH_2Cl + H_2O$ monochloramine

$NH_2Cl + HOCl \rightarrow NHCl_2 + H_2O$ dichloramine

$NHCl_2 + HOCl \rightarrow NCl_3 + H_2O$ trichloramine (nitrogen trichloride)

The relationship between the amounts of the three types of chloramine depends on the pH and the ammonia concentration of the water. As trichloramine can only form at very low pH values, the other two prevail in wastewater treatment; dichloramine is a more powerful bactericide than monochloramine. However, it is much less powerful than either hypochlorous acid or the hypochlorous ion.

8.4 RESIDUAL CHLORINE.

There is no need for a chlorine residual after disinfection has taken place. In fact, where treated effluent is discharged into a water body, residual chlorine is not desirable.

8.5 THE VARIOUS CHEMICALS USED FOR CHLORINATION.

8.5.1 Liquefied Gas Chlorine.

This is gaseous chlorine that has been liquefied under pressure and is stored in pressure cylinders. The pressure at which chlorine liquefies varies from 266 kPa at 0°C to 4 100 kPa at 100°C. The cylinders are filled to a level so that at 65°C the liquid chlorine occupies 80% of the volume of the cylinder and gaseous chlorine 20%. The cylinders are fitted with fuse plugs for the release of gas when the temperatures exceed 65°C. It is thus very important that chlorine cylinders are not stored near steam pipes, boilers etc.

Dry chlorine gas is not corrosive and may be transported in iron cylinders or piped in iron pipes. When moist, chlorine gas becomes very corrosive due to the formation of hydrochloric acid and hypochlorous acid.

CHLORINE GAS IS VERY POISONOUS! It is 2.5 times as dense as air and will sink to the floor when leakage occurs. The rooms used for storing chlorine and the chlorinator must have sealed walls and an outward opening door. The door must have air ducts near the floor level and air vents near the top. It is recommended that at the larger installation, a fan be installed to blow air into the room near the ceiling, with the usual vent at the bottom of the door.

Gas chlorinators consist of pressure-reducing valves actuated by metal diaphragms or hydraulically operated floats and orifices for measuring the rate of flow of gas after it has been reduced to a uniformly low pressure, and a device for injecting the gas into the water. Chlorine gas may be piped in tubes through a non-return valve directly into the water in which the tubing is led either to a perforated solid silver diffuser inserted into the water pipe or to a carborundum diffuser inserted into the pipe. These are the so called "dry feeders" which are designed for use where high pressure water is not available; they can be used at points where the water pressure in the pipes is less than 175 kPa.

During cold weather "chlorine ice" may form at the diffuser where the gas comes in contact with the water. Heating the diffuser may overcome this problem.

In general, the solution feed chlorinator is preferable. This also has a pressure reducing valve and a gas flow measuring device, often a rotameter, and has an injector that dissolves the chlorine and injects the solution into the water pipe.

This system needs water at a pressure of 3 times that of the water being chlorinated; often a booster pump is used.

The usual gas cylinders used in this country are 68 kg or 900 kg. The maximum draw off rate from a 68 kg cylinder is about 15 kg per day. (This is temperature dependent). A higher draw off rate will cause an excessive drop in temperature of the chlorine in the cylinder.

Corrosion of the chlorinator may be prevented by covering all exterior parts with a suitable grease such as petroleum jelly. The location of a leak can be readily detected by removing the stopper of a bottle of ammonia solution and holding the neck of the bottle near the various parts of the apparatus - if chlorine is escaping it combines with the ammonia fumes to provide a white clearly visible smoke of ammonium chloride:

$$Cl_2 + H_2O + NH_3 \rightarrow NH_4Cl + HOCl$$

The supply of gas to a chlorinator must be shut off at the cylinder and the equipment purged of chlorine through the vent before repairs are attempted. **Each time a cylinder is connected up, a new lead washer must be used.**

Chlorine should be applied to the water at a point where all the water being treated comes in contact with the chlorine. Chlorine gas should not be applied at a depth of less than 1.2 m in open conduits and flumes, as it will not be completely absorbed.

As individual chlorinators vary, the manufacturer’s instruction manual must be studied before the unit is operated.

8.5.2 Sodium Hypochlorite solution.

Sodium hypochlorite solution usually contains 3 - 15% available chlorine by mass. This solution stored in a dark, cool place is reasonably stable. Its main advantage is that it is free from suspended solids, but does have a limited shelf life. The concentrated solution is usually diluted as needed, to a strength of 1% (10 000 mg/L) available chlorine for more convenient application with solution-feed equipment.

8.5.3 Calcium Hypochlorite Granules.

This is a modern product considerably superior to chloride of lime. It usually contains about 90% available chlorine. It deteriorates more slowly than chloride of lime, particularly in hot, humid conditions. It is reasonably soluble in water, leaving some suspended matter.

8.6 OTHER DISINFECTION PROCESSES.

8.6.1 Ozonation.

Ozone (O_3) which is an allotrope of oxygen is a very powerful disinfectant.

Among the advantages of this method are:

1. no taste is imparted to the water, as a result of reaction with chemicals present in the

water;

2. some organo chlorinated chemicals are oxidised;

3. viruses are inactivated.

Among the disadvantages of this method are:

1. ozone cannot be stored and has to be produced as required;

2. it is a costly process requiring skilled operation and maintenance;

3. the ozone does not have any residual effect;

4. ozone is very corrosive and attacks most metals, only glass and certain grades of stainless steel may be used in contact with the gas.

8.6.2 Chloride Dioxide ClO_2.

This is a very powerful oxidizing agent. It is prepared by reacting chlorine with sodium chlorite.

Among the advantages of this method are:

1. no taste is imparted to the water as a result of reaction with chemicals in the water;

2. phenols are oxidized completely.

Among the disadvantages of this method are:

1. the chlorine dioxide cannot be stored and has to be prepared as required;

2. only low concentrations of chlorine dioxide can be used since chlorine will remain in the reaction mixture and will pass into the water being treated;

3. the residual sodium chlorite has no disinfecting effect.

8.6.3 Ultra Violet Rays.

Ultra violet rays are an effective disinfecting agent and also kill large numbers of viruses, provided they are applied to a thin stream of water of the order of millimetres and at sufficient power, and provided also that the lamps are changed about every year as their output diminishes with time. The water must be clear, colourless and free from turbidity; it must contain no iron, organic colloids or planktonic microorganisms likely to form deposits on the pipes, with a consequent considerable reduction in radiation.

If these conditions are fulfilled, all living cells exposed to the UV rays die or at least are no longer capable of reproducing or acting on the surrounding medium.

To guarantee reliability, the unit must be large, well-monitored and well-maintained, and operated with water of constant quality throughout the year.

A major disadvantage of this process is the lack of a residual disinfection effect and also any contamination of the water downstream will allow bacteria to grow again.

8.7 FACTORS AFFECTING DISINFECTION.

The presence of suspended solids in the effluent to be disinfected can affect the efficiency of the disinfection process. This is because organisms can be enmeshed by organic or inorganic debris and thus be beyond the reach of the disinfecting agent. Later in a river, for example, the debris can be split up and the organisms released into the water.

WASTEWATER TREATMENT.

PART 9

WASTEWATER TREATMENT PROCESSES FOR SMALL OR ISOLATED COMMUNITIES

9.1 INTRODUCTION.

In many areas, houses are so spread out that the cost of providing a sewer system to collect the wastewater is prohibitive. Here on site treatment and disposal of wastewater would be appropriate.

When an area is being developed for housing and is being sewered, some form of wastewater treatment will be required immediately unless on-site treatment, as above, is to be installed initially. An oxidation pond system (9.3) may be suitable as an initial stage, as the contributing population grew, the oxidation pond system could be upgraded into an aerated lagoon system (9.4). With a further increase in load, a biological filter or activated sludge works could be built and the oxidation pond or aerated lagoon could be used as maturation ponds to follow the main process. The floating aerators used in the aerated lagoon could be installed in an aeration tank for use in the activated sludge process.

Rotating Biological Contactors could also be used in place of biological filters or activated sludge.

9.2 SEPTIC TANKS.

A septic tank is a sedimentation tank in which the solids are allowed to remain in an undisturbed state sufficiently long to permit their disintegration by bacterial action. The action in the tank is mainly anaerobic, and is somewhat similar to that in a wastewater treatment works digester. The tank can take all the wastes from a normal household. It is advisable to install a grease trap before the septic tank. Care must be taken not to allow disinfectants to enter the tank or the bacteria which disintegrate the solid matter will be largely or entirely destroyed. Excessive volumes of water, such as during rainy periods, must not be allowed to enter the tank.

On entering a septic tank, the solids separate from the liquid with some floating but most sinking. Anaerobic bacteria metabolise the solids converting much of them to a liquid and/or a gas, and leaving a humus-like residue. This residue, together with grit and other heavy material, forms sludge at the bottom of the tank and must be removed periodically; as it reduces the working volume of the tank, and would ultimately fill it. Some of the solid matter remains in the surface scum. See Figure 42.

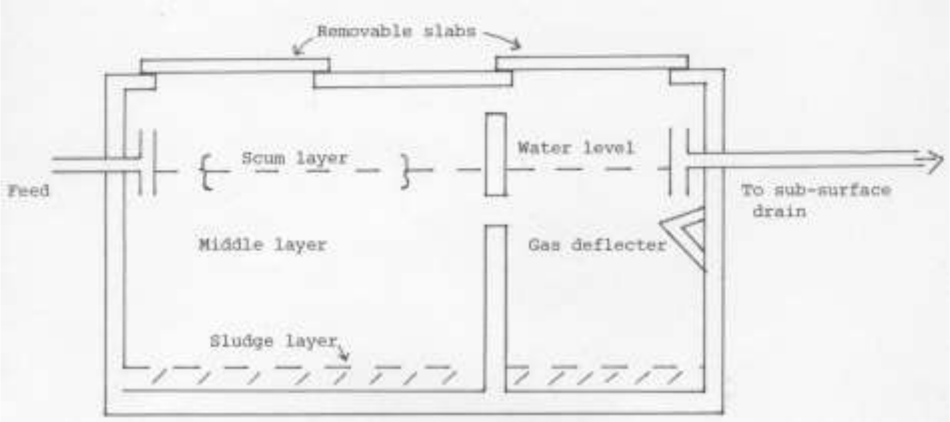


Figure 42 – CROSS-SECTION OF A TYPICAL SEPTIC TANK.

In a working septic tank there are three distinct layers:

1. the top layer of scum, about 150 mm thick, the bottom part of which is very active;
2. a large middle layer, through which particles of sludge rise and fall;
3. a bottom layer, the lower part of which becomes an inactive heavy sludge that gradually accumulates.

Gases are given off during decomposition in a septic tank, and suitable vents must be provided for these. Some of the liquid from the middle of the tank is discharged by displacement each time wastewater runs into the tank.

This liquid is still too impure to be allowed to flow away from the property and requires further treatment. It is usually irrigated through a sub-surface or French drain. Because of this need to have a sub-surface drain, the use of septic tanks should be confined to areas where adequate land and suitable soil is available for the drain. Wells and boreholes must be kept in a good distance away from such drainage areas. Sludge removed from the tank during cleaning should be buried, or dug into the surface soil as manure or added to a compost heap.

Septic tanks should be well built in reinforced concrete, or in hard brick plastered with cement mortar.

The first chamber should be a capacity of about 2/3 of the total. For a single dwelling, the septic tank will have a total capacity of about 2 000 litres.

The inlet pipe should discharge a little below the surface so as not to disturb the scum layer. Exit openings or pipes from the first chamber to the second, and from the second to the sub-surface drain should also be a little below the surface.

When wastewater in run into a new septic tank, it will start operating very slowly; as it takes time for the active anaerobic organisms to develop. During the first few weeks, the contents of the tank will be acid and will give off offensive odours. This may be controlled to some extent by the careful addition of calcium hydroxide. However, as with a digester on a wastewater treatment works, the best method of starting a tank is to add several litres of active sludge from another septic tank that is operating efficiently. Well-digested sludge from a WWTW may also be used. As a last resort well-rotted compost or manure may be used. This is called "seeding" a septic tank i.e. adding the correct anaerobic organisms.

When a tank has been emptied in order to clean it out, several litres of the active contents must be returned to the tank before the tank is put back into use.

The sub-surface drain is fed intermittently with effluent displaced from the septic tank. The media of the sub-surface drain is alternately soaked and allowed to partially dry out. In some respects it may be likened to contact beds or to intermittent sand filtration. Some nitrification may be expected in this area.

Septic tanks may also be used for hotels, groups of houses or a block of flats with populations of up to 50 persons. Where the ground is unsuitable to sub-surface disposal of the septic tank effluent, it may be treated in a Rotating Biological Contactor works.

9.3 OXIDATION PONDS (STABILIZATION PONDS).

Oxidation Ponds are usually earthen basins, open to the sun and the air. They depend on natural biological, chemical and physical processes to stabilize wastewater. They have also being called lagoon and stabilization ponds. **They must not be confused with maturation ponds. The latter are a form of tertiary treatment that follows biological filters or activated sludge**. The natural treatment processes, which may take place simultaneously, include sedimentation, digestion, oxidation synthesis, photosynthesis, endogenous respiration, gas exchange, aeration, evaporation, stratification and mixing and seepage.

Among the advantages of oxidation ponds are:

1. Their ability to handle wide variations in organic and hydraulic loading with little effect on the quality of the effluent;
2. they require minimum control by relatively unskilled staff;
3. relatively low capitals cost, if the cost of the land is not included;
4. low operation and maintenance costs.

Among the disadvantages are:

1. a large land area is required for a given organic or hydraulic loading;
2. the localised odour problem that occurs when the ponds become anaerobic either due to overload or during reduced sunshine and lower temperatures during winter;
3. the high algae content of the final effluent;
4. long retention times are necessary for nitrification to occur.

9.3.1 The Aerobic Pond.

Aerobic ponds are shallow ponds that rely on algae to provide the oxygen required for the treatment process. Ideally they contain DO throughout their liquid volume at all times (i.e. there are no anaerobic zones). However, even in aerobic ponds there will be a sludge layer at the bottom.

There will be some surface absorption of oxygen. It is possible to add floating aerators to increase the oxygen transfer into the body of the liquid. These are sometimes known as Aerated Lagoons.

Screening of the influent is preferable to remove large floating solids, before the influent enters the pond.

Where electricity is available, some works may have an algae return flow to the first pond. This would be similar (and perform the same function) as return activated sludge; to ensure good contact between the active organisms and the influent flow.

9.3.2 The Anaerobic Pond.

The anaerobic pond is much deeper than an aerobic pond. These are constructed so that anaerobic decomposition of settleable solids in the influent, can take place. Only partial stabilization takes place, so these ponds must be followed by facultative or aerobic ponds, to complete stabilization of the organic material and provide additional solids removal.

Anaerobic ponds are typically used as the first step for treatment of strong organic wastes, such as those from industries processing vegetables and fruits, meats, milk, or other foods. These ponds are not ordinarily used for treatment of domestic wastewater, although they may be applied as a first treatment step if the wastewater is abnormally strong because of industrial discharge.

Anaerobic ponds often produce a strong smell as some of the products of degradation are volatile. This property should be noted when siting the ponds.

9.3.3 The Facultative Pond.

Facultative ponds are medium depth ponds (1 - 2 m), with an aerobic zone overlying an anaerobic zone (with some sludge deposits) and a zone between the two, where facultative bacteria primarily function. Solids in the influent sink to the anaerobic zone while the soluble fraction of the influent is treated in the aerobic zone to be oxidised. Facultative ponds are sometimes called aerobic - anaerobic ponds. Most wastewater treatment ponds are facultative ponds. Facultative ponds are often used as the final ponds, if anaerobic ponds are used as the initial ponds.

9.4 THE TREATMENT PROCESS IN OXIDATION PONDS.

The organic materials, suspended dissolved in the raw wastewater or scoured from the bottom by intra-pond mixing (as a result of wind or thermal turnovers), are biochemically stabilized, usually aerobically by bacteria, algae, and to, a smaller extent, other biota. Bacteria release enzymes into the water that catalyse chemical reactions, producing simpler chemicals used to synthesize new cells and aid cell metabolism. The organic carbon is broken down by heterotrophic bacterial action, which results in the formation of urea or protein.

These are in turn broken down by heterotrophic facultative and anaerobic bacteria. The end products are ammonia, new bacterial cells, carbon dioxide and residues. Algae grow symbiotically with bacteria; the algae utilize carbon dioxide in their photosynthesis process to produce oxygen as a by-product. This is used by the aerobic bacteria. However, at night and during endogenous respiration algae also use oxygen and oxidize some of the compounds they have produced and stored during the period of photosynthesis.

The various processes that take place in a facultative pond are shown in figure 43.

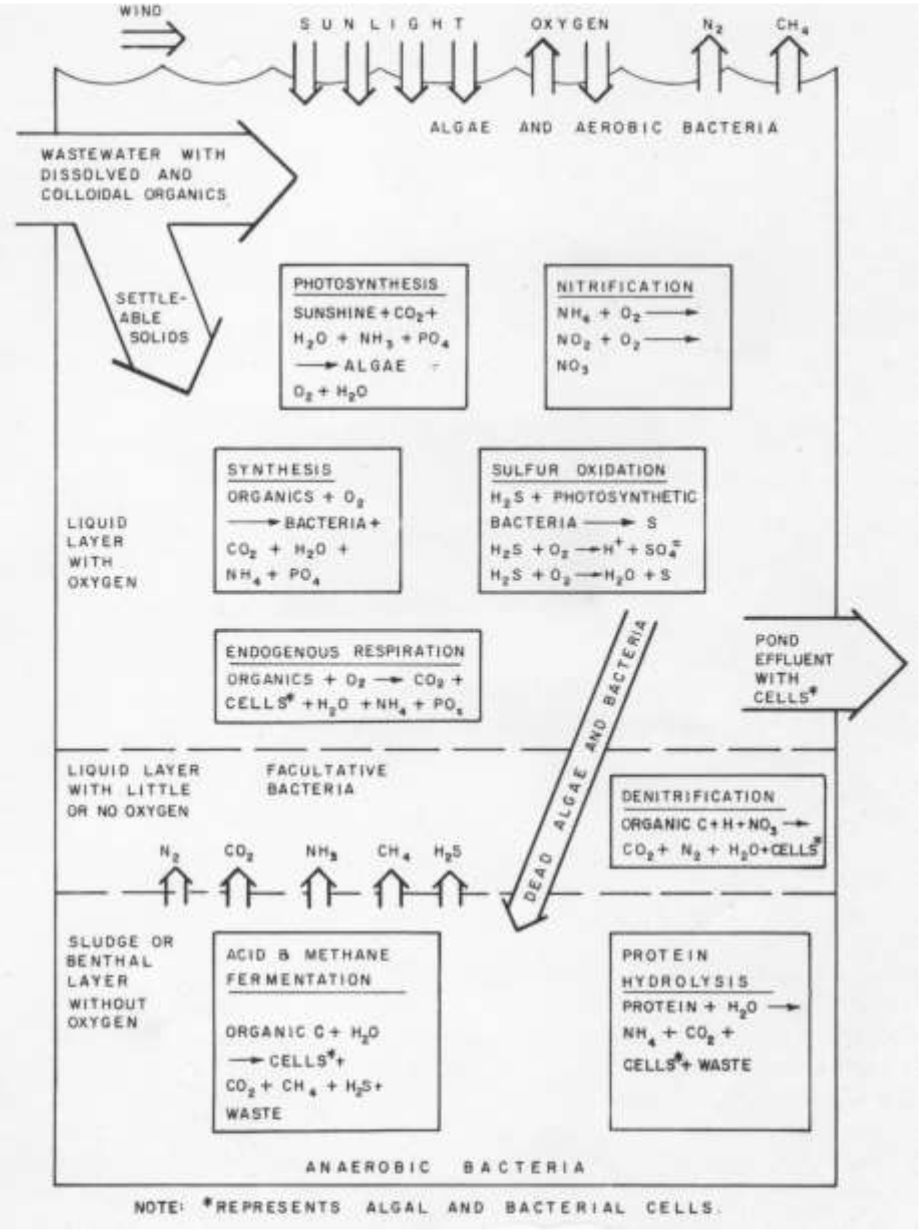


Figure 43 – THE VARIOUS PROCESS TAKING PLACE IN FACULTATIVE PONDS.

The settleable solids, including dead microbial cells, settle to the bottom of the aerobic zone into an anaerobic zone, reducing the requirements for dissolved oxygen. Such solids undergo acid and methane fermentation, hydrolysis, and other biochemical changes. These changes result in the formation of new bacterial cells, provide energy for the cells, and release methane, carbon dioxide hydrogen sulphide, ammonia, various organic acids, and residues. The ammonia and sulphurous gases that evolve in the anaerobic decomposition rise into the aerobic zone. Here nitrifying and sulphide-oxidizing bacteria convert the ammonia to nitrite and nitrate and the sulphides to sulphates and to sulphur. If liquid containing nitrate enters an anaerobic zone, denitrifying bacteria reduce the nitrate to gaseous nitrogen, which escapes to the atmosphere. In some cases, nitrogen-fixing bacteria convert gaseous nitrogen to nitrates.

Dead algal and bacterial cells undergo lysis, or disintegration, in the same manner as other dead organic matter and in the process, create an oxygen demand. On the other hand, the products of disintegration, oxygen and carbon dioxide provide nutrients for the syntheses of new algal cells and the synthesis continues until the synthesis-decay-sedimentation cycle leads to removal of most of the BOD and suspended solids from the wastewater. The optimum temperature for efficient functioning of treatment ponds is 20ºC to 25ºC. At lower temperatures, bacterial action is reduced.

The performance of all wastewater treatment ponds is particularly affected by the following factors:

1. the organic loading per unit area;

2. the air and water temperature;

3. mixing caused by wind action;

4. the actual retention time in the ponds;

5. the sunlight energy.

As with the biological filtration and activated sludge processes, nitrification occurs only with lightly loaded ponds. As the energy of the sun plays a major role in the activity of the algae to produce oxygen, the actual loading rate needed to ensure nitrification cannot be stated with such accuracy as with the aforementioned processes.

Experience has indicated that best results are obtained by using oxidation, ponds in series, the use of 4 ponds is recommended. The optimum depth of **aerobic** ponds is about 1.2 m. Weeds grow readily in ponds that are too shallow. Sunlight does not penetrate to the depths of deeper ponds and stratification can occur. For the treatment of unsettled wastewater, total retention period in the ponds of 20 - 35 days, is required. The retention periods of the 4 ponds should be in the ratios of 50% : 25% : 12.5% : 12.5%.

It is recommended that the raw wastewater enter the first pond at two or more places to distribute the load. It is distinctly advantageous to recirculate algal-laden water from the second or third pond to the points where the raw wastewater enters the first pond. This recirculation has a similar effect to that used in the activated sludge process in that it ensures that enough of the active organisms, here bacteria and algae, are in contact with the wastewater.

The efficiency of oxidation ponds is reduced in winter as a result of the lower temperatures and the shorter daily periods of sunlight. The dissolved oxygen content of the water may be raised by the use of floating surface aerators. Some examples are shown in figure 44.

The inlet to the ponds should be below water level and the outlet should have a scum board to prevent scum passing into the next pond. When a pond system is started, the problem may arise that the initial flow is insufficient to fill the first pond rapidly. When the seepage and/or evaporation is excessive then much of the liquid fraction is lost leaving the solids to dry out on the surface on the pond floor and fly breeding may result. The floor of the pond will in time seal up and seepage will reduce. If extra water can be introduced into the system, this will help reduce start-up problems.

The first pond may be sub-divided into smaller areas by low walls, so that each section fills one at a time.

Since it is essential to prevent plant growth around the ponds in order to avoid mosquito breeding, it is recommended to place concrete slabs on the banks extending down about 300 mm into the water. The subsequent saving in labour and nuisance is likely to offset the additional cost of cement paving. The slabs also provide erosion protection.

The removal of algae, since algae are very small, also since they tend to be kept suspended because of minute oxygen bubbles, formed by photosynthesis, remaining attached to them; they cannot be readily be removed by settlement. Algae can be removed by chemical precipitation of by dissolved-air flotation but these are expensive and need mechanical plant.

9.5 AERATED LAGOONS OR PONDS.

When existing facultative or aerobic ponds become overloaded, or a higher level of treatment is required; aeration facilities may be added to meet the pond's oxygen requirements. With aeration, a pond's treatment capacity may be increased by up to a factor of up to 5. The effect of the lower temperatures and less sunlight in winter is minimized. It may be necessary to aerate only during winter or during the night only, in order to save electricity.

The various types of aerators that may be used are shown in figure 44.

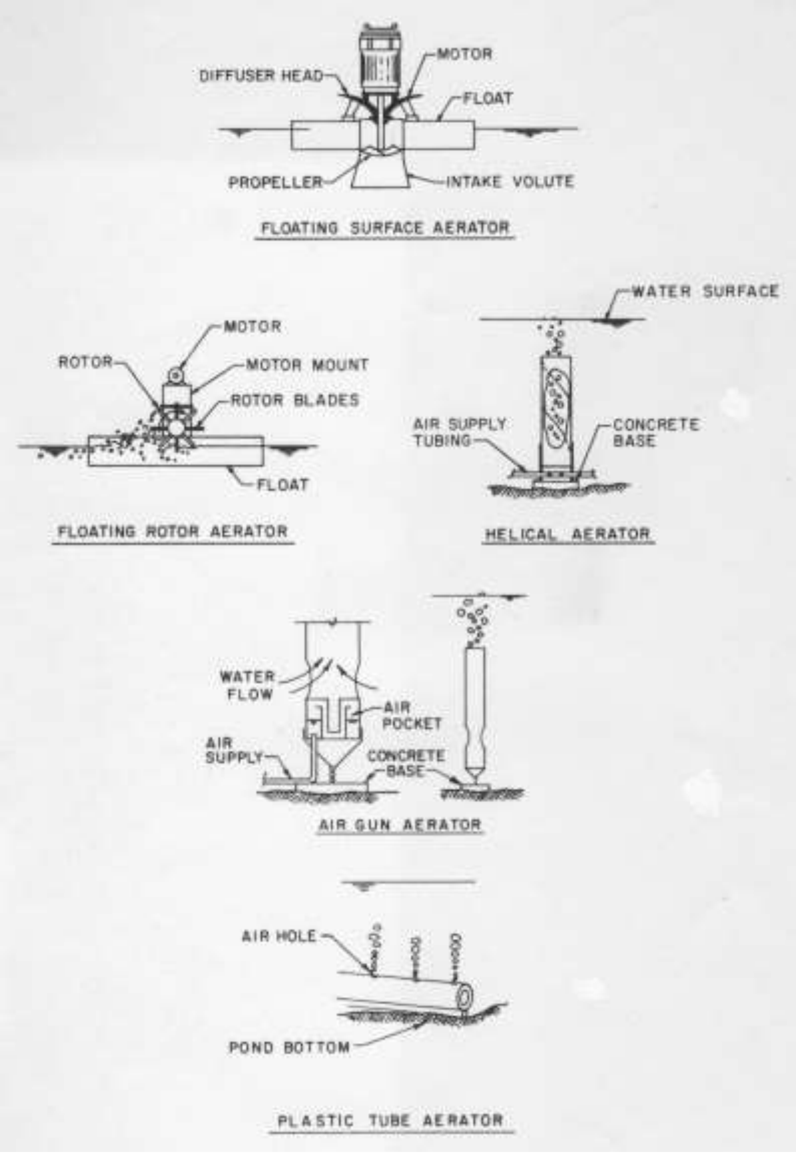


Figure 44 – VARIOUS TYPES OF AERATION DEVICES FOR OXIDATION PONDS.

9.6 EFFLUENT QUALITY.

In all the ponds systems described above, there will be periods when the final effluent contains a significant concentration of algae. The algae may have to be removed in order to comply with the requirements of the standards. Algae can be removed by sand filtration, dissolved air flotation, land treatment, precipitation with sedimentation etc.

TABLE 7 - TYPICAL EFFLUENT QUALITY FROM OXIDATION PONDS.

PARAMETER	TYPICAL EFFLUENT QUALITY
BOD	5 - 25 mg/L
COD	25 - 100 mg/L
Ammonia Nitrogen	0 - 5 mg/L
Suspended Solids	20 - 40 mg/L
E. Coli	1000 - 10 000 MPN/100 mL

9.7. TYPICAL REMOVALS AT END OF SECONDARY TREATMENT STAGE.

At the end of the secondary treatment stage, the bulk of the pollutional material (measured as the COD) will have been removed, and if a low rate process is used, the ammonia content should be less than about 5 mg/L

A typical analysis will be of the secondary sedimentation tank effluent would be:

TABLE 8 - TYPICAL % REMOVAL OF SELECTED PARAMETERS:

PARAMETER	TYPICAL % REMOVAL
BOD	90
COD	85
Ammonia Nitrogen	90
Suspended Solids	90

Should a higher quality effluent be required, them tertiary treatment will be necessary.

WASTEWATER TREATMENT.

PART 10

TERTIARY TREATMENT.

10.1 INTRODUCTION.

Generally the effluent from the secondary sedimentation tank of a medium or low rate biological filtration or activated sludge works will meet the requirements of the General Standard of the Department of Water Affairs. In certain circumstances e.g. when the Special Standard is required or when it is intended to use the treated effluent for the irrigation of playing fields etc., further treatment is required.

Among the methods used are:

1. Maturation Ponds;

2. Micro straining;

3. Sand Filtration;

4. Land Treatment.

10.2 MATURATION PONDS.

The difference between these and Oxidation or Stabilization ponds must be stressed. Maturation ponds are a tertiary treatment process and follow a Biological Filtration or Activated Sludge Works, whereas Oxidation or Stabilization ponds are themselves a secondary treatment process.

Usually the effluent from a conventional works passes into a series of ponds which discharge into the receiving waters. During the passage through the ponds, small particles of sludge lost from the secondary sedimentation tank settle to the bottom. Maturation ponds reduce the bacteriological count of the effluent to a very large degree. This is by settlement of solid particles containing absorbed and trapped bacteria and by natural die-off; the latter is hastened by the sunlight. Helminth ova also settle out. Generally some nitrification and denitrification takes place. Phosphates tend to precipitate out at the higher pH values, and settle to the bottom deposits.

Another very important feature of the maturation pond is that it gives the effluent a chance to absorb oxygen from the air. The dissolved oxygen content in the last pond should be high enough for fish to survive and it is advantageous to stock as many ponds as possible with fish to reduce the mosquito population. The growth of algae can be a problem, but if little carryover of sludge occurs from the secondary sedimentation tank there should not be enough nutrient remaining in the effluent for heavy algal growth to develop.

Retention periods of 10 to 20 days are recommended, but are not always possible due to the large areas of land required. Maturation ponds balance the load passed onto the receiving waters and act as a very efficient buffer in case of overflows or interrupted operation of the works.

10.3 MICRO STRAINING.

In areas where land for maturation ponds is too expensive or not available, micro straining may be used to remove the fine suspended solids carried over from the secondary sedimentation tank. A micro strainer consists of a drum, closed at one end, revolving on a horizontal axis and covered with stainless steel or synthetic fabric of special weave. The drum is mounted in a tank which is divided into two components, one for the influent and the other for the strained effluent. Secondary sedimentation tank effluent enters the open end of the drum and passes out through the fabric, the solids being retained on the inside of the drum. As the drum rotates these solids are under pressure through a row of jets fitted on top of the machine and extending the full width of the drum. Wash water containing the removed solids matter is collected in a trough inside the drum and is returned to the inlet channel of the works. Provided that the secondary treatment is adequate, micro-strainers produce a consistently good effluent, but problems are experienced when severe bulking in an activated sludge causes losses of much sludge from the secondary sedimentation tank.

10.4 SAND FILTRATION.

These filters are much the same as those found in water treatment. Sand filters were covered in detail in the ***Process Controller's Guide to Water Sources and Water Treatment.*** (Volume 2).

10.4.1 The Slow Sand Filter.

This is not often used in modern wastewater treatment works because of the large area of land required and the relatively high maintenance costs (mainly labour). Surface loading rates are about 3 - 8 m^3/m^2-d. The sand depth is 150 - 500 mm, and the effective particle size is usually about 0.3 mm with a uniformity coefficient of about 2.5.

10.4.2 The Rapid Sand Filter.

This is the most common type of filter used in wastewater treatment. Filters are usually designed on loading rates of 150 - 250 m^3/m^2-d, the sand depth usually about 1.2 m. The effective particle size is about 0.45 mm with a uniformity co-efficient of 1.8 or less.

10.4.3 The Pressure Sand Filter.

This operates on the same principle as rapid sand filters except that the water is pumped into the filter chamber instead of relying on the head above the sand. They have the advantage that the discharge the discharge pressure from the filter may be any reasonable value whereas the discharge pressure from a rapid sand filter is atmospheric pressure - this means that it is possible to pump the water through the filter and to deliver the filtered water at a level far above that of the filter itself. A disadvantage of this system is that the backwashing process cannot be watched. It is necessary to inspect the backwash for sand being carried out of the filter.

10.5 LAND TREATMENT.

Two different methods are in use:

10.5.1 Irrigation over grassland.

With small treatment works where there is suitable land available, irrigation of the effluent over grass plots is a cheap and effective method of removing solids. The land is divided into two or three plots, one of which is in use at a time. The treated effluent is run onto the plots through a system of channels, flows over the surface, and is collected by a second series of channels. The grass needs cutting occasionally but should not be kept short and periodically the plots must be cleaned and accumulated solids remove. Some nitrogen and phosphorus removal will take place.

10.5.2 Intermittent Sand Filtration.

This is usually only used in sandy areas where beds are constructed with earth walls to separate each bed. Underdrains to collect the filtered effluent are laid where it is desired to discharge the treated effluent into a lake or river. These are omitted when the water is to be allowed to recharge an underground aquifer. The whole secret of this process is to flood the beds with a thin layer of water, about 50 mm, once or twice per day - hence the use of the term intermittent.

The rest period is vital to the well-being of the land. Surface accumulation of solids needs to be removed from time to time.

WASTEWATER TREATMENT.

PART 11

ADVANCED TREATMENT.

11.1 INTRODUCTION.

The tertiary treatment methods mentioned above have certain limitations when it comes to high quality requirements for particular re-use purposes, or to removed nutrients, toxins, etc., as a safeguard against pollution in receiving waters into which the effluents may be discharged. In these cases, one needs to use advanced treatment methods. These methods consist of physical and chemical techniques.

There are two main areas of advanced treatment:

1. removal of inorganic ions;

2. removal of organic compounds only.

11.2 REMOVAL OF INORGANIC IONS.

In virtually all industrial and domestic uses of water there is an increase in the total dissolved solids content and particularly in the inorganic dissolved solids content. This mineralization of water is becoming of concern because ions such as sodium, potassium, chloride and sulphate pass through the preliminary, primary, secondary and tertiary stages without being removed. In virtually all inland water systems, water is abstracted, treated, used, treated again and then discharged back into the river either directly or via infiltration areas near the rivers. As one passes along a river from source to sea discharge, this abstraction and discharge cycle may be repeated many times. Eventually the water may become so mineralized that it can no longer be used.

The actual increase in dissolved solids per usage cycle can vary very widely from 200 mg/L in high class residential areas to 1 000 mg/L in certain industrial processes.

Much can be done to reduce mineralization by prohibiting the discharge of saline wastes into the sewers. Many Local Authorities have imposed a limit of 1 000 mg/L of inorganic dissolved solids on industrial effluents. This is a tough limit but it is necessary to enable the Local Authority to comply with the General Standard requirement of an increase of not more than 500 mg/L above the intake water.

Many industrial processes use sodium salts, particularly sodium chloride. If some of the treated effluent is to be used for irrigation; then sodium chloride is just about the worst chemical to add. When the sodium level is too high, the quality of the irrigated soil can be affected deleteriously as ion exchange reactions take place. Sodium is taken up in the soil while calcium and magnesium as released and are leached out. This causes a deficiency of the latter two minerals to occur and the high sodium content adversely affects the soil structure. A high dissolved solids content in the irrigated water can cause leaf burn when evaporation of the droplets of water on the leaf leaves a little crust of salt.

11.3 REMOVAL OF DISSOLVED SOLIDS FROM WATER BY ION EXCHANGE.

This is a two stage process. In one stage a cation exchange resin is used and cations such as sodium, calcium, magnesium, etc., are removed from the water and replaced by the hydrogen ion. It has been seen that the pH of the water will drop due to the production of H^+ ions. This partially treated water is then passed into an anion exchange resin where sulphate, chloride are removed and replaced by hydroxide ions (OH^-). The hydrogen ions and hydroxide ions combine to form water and the pH will tend to move towards neutrality. Carbonates will be removed by strongly basic resins but not by weakly basic resins.

Ion exchange columns are either mixed bed where the cation exchange resin is mixed with the anion exchange resin or kept in separate units. When being regenerated, the resins in the mixed bed unit must be separated so that they may be treated with the separate chemical solutions. Usually the resins have differing densities so that they may be separated by backwashing.

The cation exchange resin is regenerated using sulphuric acid and the anion exchange resin using sodium hydroxide. Both cationic and anionic resins are available with capacities of up to 60g hardness as $CaCO_3$ per litre; i.e. about 1.2 eq/L.

The amount of sulphuric acid is needed for regeneration is about 3 g H_2SO_4 per g hardness removed as $CaCO_3$. The amount of sodium hydroxide needed is also about 3 g NaOH per g hardness removed as $CaCO_3$. Usually 10% solutions are used.

The reactions are as follows:

$Na^{++} + R_cH \rightarrow R_cNa + H^+$ R_c = cation exchange resin

$Cl^- + R_aOH \rightarrow R_aCl + OH^-$ R_a = anion exchange resin

The regeneration reactions are as follows:

$H^+ + R_cNa \rightarrow R_cH + Na^+$ R_c = cation exchange resin

$OH^- + R_aCl \rightarrow R_aOH + Cl^-$ R_a = anion exchange resin

The brine formed from the sodium chloride and other salts from the regeneration needs to be disposed of without causing pollution. Disposal of brine wastes can be a serious problem.

Demineralized and untreated water are blended to give the required hardness to reduced operating costs.

The cost of this process at present usually exceeds that of distillation when the total solids content is greater than about 3 000 - 5 000 mg/L.

The effluent from the unit in the case of the mixed bed system or from the anion exchange unit in the case of the separate bed system is checked for electrical conductivity at intervals. When the conductivity starts to rise then breakthrough is occurring because the ion exchange capacity is exhausted and regeneration is needed.

This process is operated on a batch wise basis, this is a disadvantage. A recently developed process is almost continuous flow where a small quantity of resin is removed from the bottom of the ion exchange column during a short shutdown period. The resin is then regenerated in a separate column while the main column is producing demineralised water. It is called the counter current continuous flow ion exchange system CCIX. Because the anion exchange resin is regenerated in a separate container, it is possible to use calcium hydroxide slurry as a regenerant. With the use of sulphuric acid as a regenerant for the cation exchange resin, a precipitate of calcium sulphate is formed. This may be removed by sedimentation.

The reactions are as follows, using calcium as an example:

$Ca^{++} + 2\,R_cH \rightarrow (R_c)_2Ca + H^+$ R_c = cation exchange resin

$Cl^- + R_aOH \rightarrow R_aCl + OH^-$ R_a = anion exchange resin

On regeneration using sulphuric acid for regenerating the cation exchange resin and sodium hydroxide to regenerate the anion exchange resin, the reactions are:

$(R_c)_2Ca + H_2SO_4 \rightarrow 2R_cH + CaSO4$

$$2\ R_aCl + Ca(OH)_2 \rightarrow 2\ R_cOH + CaCl_2$$

As calcium sulphate is soluble to about 2 000 mg/L, the excess will precipitate and may be removed by sedimentation.

Among the advantages of the counter current continuous method over the older batch wise method are:

1. that the process is almost continuous;
2. a much smaller excess of regenerant is required;
3. lime may be used instead of sodium hydroxide, this is cheaper;
4. some of the removed dissolved solids is precipitated and may be removed as sludge;
5. if ammonia and nitric acid are used as regenerants, then a waste ammonium nitrate solution containing up to 17% NH_4NO_3 can be produced - this has commercial value;
6. the columns are less prone to block as they are counter current washed;
7. the water recovery rate is higher.

11.4 REMOVAL OF DISSOLVED SOLIDS FROM WATER BY ELECTRO-DIALYSIS.

Special membranes are available that have the property of permitting the passage of ions without allowing water to pass through. One type of membrane permits the passage only of positive ions or cations; a second type permits negative ions, or anions to pass through.

The electro-dialysis unit consists of a series of chambers including alternating anion-permeable and cation permeable membranes, together with inert electrodes in the outermost compartments. Water introduced into alternate chambers is demineralized by passage of a direct current through the battery of compartments. The applied voltage drives anions toward the inert anode (the positively charged electrode) and out of the cells with anion permeable membranes on the anode side. However, the anions are trapped in the adjacent cell because it has a cation permeable (anion impermeable) membrane on the side facing the anode. In this way, cells from which anions migrate toward the anode also lose cations towards the cathode (the negatively charged electrode), while alternate cells retain both anions and cations. As the water to be treated is fed into the ion-losing compartments and the concentrated brine is bled from the ion-gaining compartments, the water can be de-mineralized electro-chemically in continuous flow. The overvoltage at the electrodes and the ohmic resistance of the membranes reduce the electrical efficiency of the system. The power loss may be reduced by partially demineralising the water to a T.D.S. of about 500 mg/L only. The higher the salinity, the higher the electrical costs. Usually water of more than 5 000 to 10 000 mg/L T.D.S. is best demineralized by some other method.

Electro-dialysis has the following disadvantages:

1. the feed water must be free of turbidity;
2. the iron content of the feed water must be well below 1 mg/L;
3. problems with precipitation of calcium carbonate can occur in hard waters;
4. the final water has a relatively high dissolved solids of about 500 mg/L;
5. for dissolved solids of greater than about 5 000 mg/L, it may be necessary to operate two units in series.

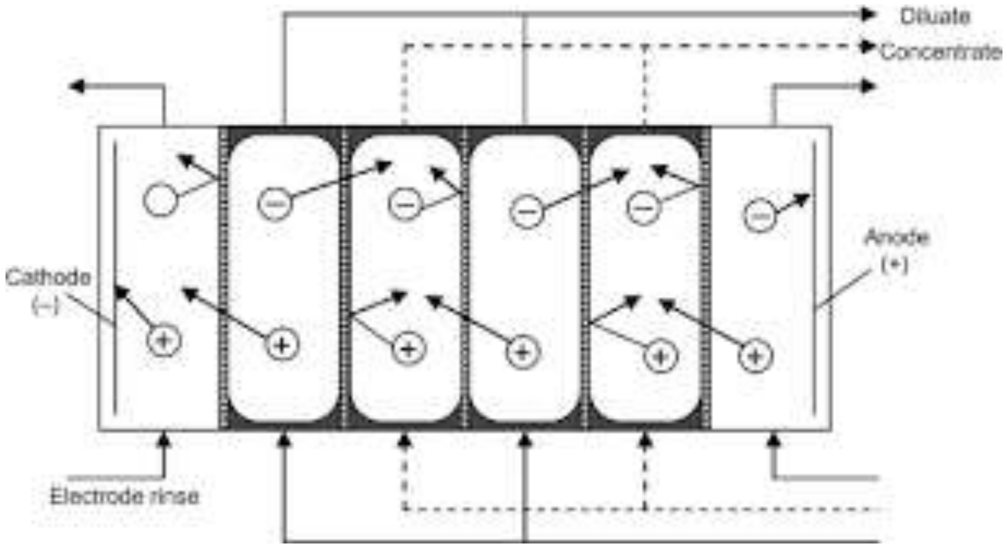


Figure 45 – TYPICAL ELECTRO-DIALYSIS UNIT.

11.5 SEPARATION OF WATER FROM THE SALT SOLUTION BY REVERSE OSMOSIS.

Osmosis is defined as the spontaneous transport of a solvent (water) from a dilute solution to a concentrated solution across an ideal semi-permeable membrane which impedes the passage of the solute (usually salts) but allows solvent flow. As a result of this flow of solvent a pressure called the osmotic pressure, is built up on the concentrated solution side of the membrane. If an external pressure, of magnitude greater than the osmotic pressure, is exerted on the concentrated solution side then the solvent will pass through the membrane to the dilute side. It can be seen that a **reverse** osmosis process has taken place. As the concentration increases, so will the osmotic pressure until it equals the externally applied pressure after which the amount of solvent passing each way through the membrane will be equal.

As with all membrane processes, the feed water has to be essentially free of turbidity. After pre-treatment, the feed water passes through a high pressure pump into the membrane modules. The concentrated reject water will pass through a pressure reducing valve to waste while the product water passes out of a second set of pipes. It may be possible to pass the reject water directly into a second module in series and only then to pass it through a pressure reducing valve, in order to improve the recovery rate. The pre-treatment stage may include pH adjustment by means of acid addition, to prolong the life of the membranes and to prevent the precipitation of calcium and iron salts.

Small molecules such as sodium and chloride do pass through the membrane to a limited extent. The rejection rate of these two ions is about 80 - 85%. The larger ions such as calcium and sulphate may be rejected to the extent of greater than 99%. Organic molecules of molecular mass greater than about 200 are almost entirely rejected. For a typical brackish water, the overall salt rejection is about 90%.

Fouling of the membranes is a problem, both from the cleaning aspect and the reduction in rate of yield.

Reverse osmosis may be used for waters of up to about 10 000 mg/L dissolved solids. The product water dissolved solids is usually of the order of 500 mg/L with an 80% water recovery. At lower rates of water recovery, a higher quality water may be obtained.

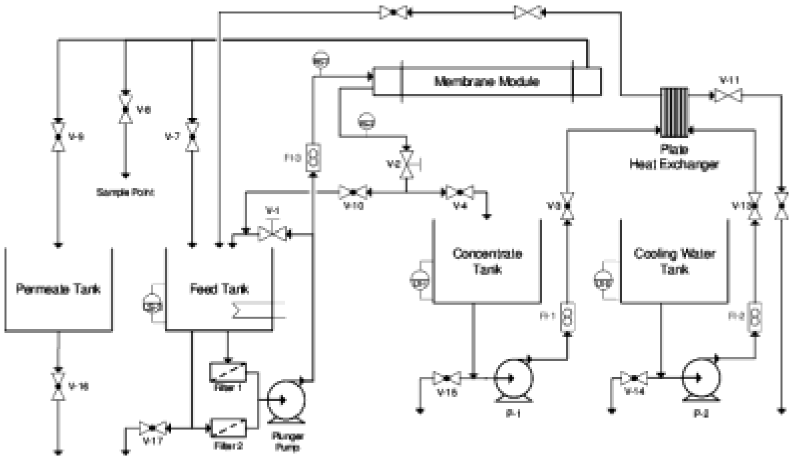


Figure 46 – TYPICAL LAYOUT OF REVERSE OSMOSIS UNIT.

11.6 SEPARATION OF THE WATER FROM THE SALT SOLUTION BY ULTRAFILTRATION.

Ultrafiltration is similar to reverse osmosis in that it depends on a pressure driving force and a membrane that is permeable to some components in the water and impermeable or only slightly permeable to others. The main difference is that ultra-filtration operates at relatively low pressures of the order of 500 kPa. Ultrafiltration is mainly used for removal of organic materials with molecular masses of greater than about 500.

The rejection of low molecular substances such as sulphates is only about 10%.

11.7 REMOVAL OF ORGANIC COMPOUNDS WITH ACTIVATED CARBON.

Certain materials have the property of being able to adsorb certain substances onto themselves. The **adsorption** phenomenon is the interface accumulation of a substance. **Absorption** by comparison is a process in which one substance penetrates into another. For simplicity, one may consider **adsorption as a surface effect** and **absorption as a body effect**. Activated carbon is one material that illustrates the first effect, while the dissolving of oxygen in water, as in the aeration process, illustrates the second effect.

The activated carbon used in tertiary treatment is usually prepared from wood, coal, bone, petroleum residues or nut shells. The raw material is generally activated in an oxygen deficient atmosphere at a temperature of between 300°C and 1 000°C.

In part, the adsorption properties of activated carbon are due to its extremely large relative surface area, 200 to 1000 m^2 per gram. Activated carbon has a preference for organic compounds and, because of this selectivity, is particularly effective in removing organic compounds from aqueous solution. When the activated carbon has been exhausted, it may be regenerated and re-used.

Activated carbon has four main uses:

11.7.1 Removal of Colour.

Certain colours imparted to water by various organic materials cannot be removed easily by coagulation. Chlorination or some other oxidation reaction may also not be feasible or desirable. Often activated carbon may be used to adsorb the residual colour.

11.7.2 Removal of Tastes and Odours.

Some tastes and odours cannot be removed entirely by aeration and chlorination may enhance the taste. This is the case with phenol, where the chlorinated derivatives of phenol impart a stronger, more unpleasant taste than the original phenol itself. Activated carbon may be used to adsorb the residual taste and odour.

11.7.3 Removal of Residual Organic Material.

Certain organic materials such as volatile halogenated hydrocarbons (VHH), and pesticides are present in small amounts in some waters, or may be formed during the chlorination process. These materials cannot be removed by the conventional treatment processes. Although not much is known about the long term effects of ingesting these organic materials, they are considered undesirable. Activated carbon is effective in removing them.

11.7.4 Removal of Residual Chlorine or De-chlorination.

Activated carbon is effective in removing all forms of residual chlorine. Activated carbon may be used for any or all of the above uses simultaneously.

11.8 REMOVAL OF ORGANIC COMPOUNDS WITH OZONE.

Ozone may be used as a disinfectant either alone or in combination with chlorination. The effectiveness of ozone as an oxidizing agent goes further than just as a disinfectant.

Ozone can oxidize certain materials that are resistant to oxidation with chlorine. It does not have the disadvantage of chlorine of forming volatile halogenated hydrocarbons, actually ozone will oxidise some organo-chlorinated compounds. Ozone will oxidise phenols to tasteless and odourless compounds. Ozone destroys all polycyclic hydrocarbons such as 3:4 benzopyrene, which are suspected of being carcinogenic. Ozone also decomposes a proportion of the non-biodegradable fraction of detergents.

Ozone must be manufactured as required as it cannot be stored. It is a gas that is slightly soluble in water. It is made by passing dry air between two electrodes with a potential difference of between l0 kV and 20 kV AC. The treated air contains about l0 to 20 g ozone per cubic metre of air. Because of the low solubility of ozone in water, careful design of the ozone contact tank is required.

11.9. THE RECHARGE OF AQUIFERS USING TREATED WASTEWATER.

In land treatment (section 10.5); the effluent was collected and removed for use elsewhere or for discharge into the receiving water. If the effluent quality is high enough and the soil is suitable it may be possible to recharge the groundwater with the treated effluent. In order to prevent the pollution of the groundwater, it is necessary that the heavy metal and nitrogen contents be very low. The dissolved solids content should be of the same order, or preferably lower than the groundwater. The intermittent sand filtration method is usually the most suitable.

As the available sources of suitable water become fully utilized, one is progressing closer to the closed loop recycle system where the effluent from the advanced wastewater works is passed directly into the water reticulation system. However, there are always going to be objections to this system. A suitable variation of this system may be to recharge an aquifer with the treated water and to abstract water some distance away.

The recharge of a groundwater aquifer with treated wastewater has been practised in the Western Cape town of Atlantis for more than 30 years.

The effluent from the domestic wastewater treatment works together with storm water is polished in a reed-bed pond and then used to recharge the aquifer upstream of the potable water abstraction boreholes. The treated effluent from the industrial wastewater treatment works together with the storm water from the noxious trade area and the acid waste from the water softening plant; is used to recharge the aquifer downstream of the potable water abstraction boreholes. This also acts as a seawater intrusion barrier.

The author motivated the concept and was responsible for the management of the system for many years.

WASTEWATER TREATMENT.

PART 12

SLUDGE TREATMENT - THICKENING PROCESSES.

12.1 INTRODUCTION.

The principal end products of the wastewater treatment process are the treated effluent and one or a number of sludges. Depending on the particular requirements the effluent will have attained the required standard and may be discharged. The sludges, however, will require further treatment. The nature of the treatment required will depend on the type of sludge and also on the method used for the final disposal of the sludge.

The treatment and disposal of wastewater sludges is the most complex problem faced by the wastewater treatment works Process Controllers. Most sludges contain large numbers of micro-organisms, many of which are pathogenic (i.e. disease causing) and potentially hazardous to humans. Sludge treatment and disposal is further complicated by its variable properties and relatively low solids concentration (usually 0.5 to 5.0% before thickening).

The need for satisfactory operation of the sludge treatment and disposal system cannot be over-emphasised. The poor operation or overloading of the sludge treatment facility can lead to the return of significant quantities of organic load to the main part of the works which can cause overloading with a resultant sub-standard effluent or, even in the extreme, to process failure.

The various types of sludges that will be considered are:

1. Primary Sedimentation Sludge;
2. Chemical Precipitation Sludge;
3. Humus Sludge from Biological Filters;
4. Waste Activated Sludge;
5. Mixtures of the above.

The thickening of the sludges will be covered as will the various methods used to reduce the mass of sludge and to improve its stability.

12.2 PRIMARY SEDIMENTATION SLUDGE.

This sludge contains most of the settleable solids present in the raw wastewater and up to about 60% of the suspended solids present - depending on amongst others, the retention period in the primary sedimentation tank. (The above assumes that no sludge streams are fed back to the head of the works.)

This sludge usually contains about 3 to 5% m/v (mass in volume) of solids. The quantity of sludge produced per population equivalent (110g COD) is about 50g dry solids. One megalitre of normal domestic wastewater will produce about 4 cubic metres of primary sludge.

This primary sludge is the most unpleasant of all sludges to handle due to its instability and rapidly developing odour, high concentration of pathogenic organisms and appearance.

12.3 CHEMICAL PRECIPITATION SLUDGE.

The nature and relative quantity of this sludge varies widely, depending on the types and quantities of coagulants used. This sludge may contain from 70 to 90% of the influent suspended solids depending on the degree of chemical treatment. The sludge will contain from 2 to 5% m/v solids.

The actual mass of sludge produced will be greater than the mass of suspended solids removed due to the presence of the coagulant. This increase in mass can vary from about 10% when small doses of an iron or aluminium salt are used, up to about 100% when large doses of lime are used. This increase in mass can adversely affect the various treatment stages if not allowed for in the initial design.

12.4 HUMUS SLUDGES FROM BIOLOGICAL FILTERS.

Most of dissolved organic solids and colloidal solids as well as much of the suspended solids in the settled wastewater applied to biological filters is rendered settleable by absorption and incorporation into the zoogleal film on the filter medium. The film itself is modified by decomposition before it is sloughed off and carried out of the filter with the effluent. The actual destruction and loss of the biological film vary with the length of storage in the filter. This process may be compared with the reduction in sludge mass by endogenous respiration in the activated sludge process. Typical values are 30% in low rate filters and 10% in high rate filters. About 95% of the remaining suspended solids are captured in the secondary sedimentation or humus tank as humus sludge.

The amount of sludge produced per population equivalent of 110g COD varies between 25 and 35g depending on the loading rate. The sludge will have a solids content of about 5% m/v.

12.5 WASTE ACTIVATED SLUDGE.

It was seen that in order to keep the total mass of sludge in the system essentially constant, it is necessary to remove a certain mass of sludge at regular intervals. Ideally, the wasting should take place continuously, but batch wise wasting is usually practised. The sludge wasted will consist of suspended present in the raw wastewater if primary sedimentation is not employed or suspended solids carried over from the primary sedimentation stage, together with the zoogleal biomass produced from the metabolism of soluble materials in the feed wastewater. The biomass will consist of both live and dead organisms. The ratio will depend on factors such as the sludge age at which the system is operated.

The mass of sludge produced per population equivalent of 110g COD will vary depending on the exact conditions of the process; typical values are as follows:

1. with primary sedimentation and a sludge age of about l0 days - 32g
2. with primary sedimentation and a sludge age of about 25 days - 25g
3. without primary sedimentation and a sludge age of about 10 days - 50g
4. without primary sedimentation and a sludge age of about 25 days - 40g

The solids content of the sludge wasted will vary between 0.3 and 2% depending on the mixed liquor suspended solids content chosen and whether mixed liquor or sedimentation tank underflow is wasted.

12.6 MIXED PRIMARY - HUMUS SLUDGE.

Usually the underflow from the humus tank is passed to a point upstream of the primary sedimentation tank. A mixture of humus sludge and the settleable solids already present in the raw wastewater will then settle together to give the mixed primary - humus sludge. It has been suggested that the adding of the humus sludge upstream of the P.S.T. aids the flocculation of the solids in the raw wastewater and this in turn increases the settleability and hence removal of these suspended solids - but this is debatable.

This method is usually adopted for convenience and simplicity. Usually some excess water is removed from the humus tank when the humus sludge is extracted. If this water is then returned upstream of P.S.T. then the excess water is of no significance - provided the volume is not excessive.

The mass of mixed primary - humus sludge produced per population equivalent will vary between 65g with low rate biological filters and 75g with high rate biological filters. This sludge will usually contain 3 - 6% m/v of solids.

12.7 MIXED PRIMARY - WASTE ACTIVATED SLUDGE.

Excess activated sludge is wasted from the aeration tank (biological reactor) or from the underflow of the secondary sedimentation tank. This sludge may be passed to the head of the works where it is allowed to settle with the primary sludge. The comments relating to the alleged flocculation effect are applicable here as well.

It has been suggested that it is not good practice to co-settle the two sludges as waste activated sludge has poor compacting properties when compared with pure primary sludge. The addition of the waste activated sludge does reduce the solids content of the mixed sludge being withdrawn from the primary sedimentation tank. The degree being dependent on factors such as the relative volume of the waste activated sludge and its compactibility.

Many Process Controllers recommend the separate thickening of the waste activated sludge by one of the methods to be covered later, before the blending of the two sludges and some even recommend the separate handling of the two sludges.

12.8 MIXED CHEMICAL PRECIPITATION - HUMUS OR ACTIVATED SLUDGE.

To relieve temporary or permanent overloading or to improve phosphorus removal, some treatment works have coagulants added upstream of the primary sedimentation tank. This improves the suspended solids removal in the primary sedimentation tank and so reduces the load on the secondary treatment units. Again the quality and quantity of the sludge will depend on the types and quantities of coagulants used.

12.9 THICKENING OF SLUDGES.

It has been seen above that the solids content of the sludges discussed varies between 0.5% and 5.0%; looking at this from the other direction, these sludges contain 99.5% and 95.0% respectively of water!

One ML of raw wastewater with a suspended solids content of 400 mg/L will produce about 4m^3 of primary sludge at 5.0% solid content; assuming a 50% suspended solids capture in the primary sedimentation tank.

If the same one Ml is treated (without primary sedimentation) using the activated sludge process with a 15 day sludge age; the volume of sludge produced daily will be about 26 m^3 at 0.5% solids content.

In order to reduce the volume of the sludge to be treated, the sludge may be thickened. Here thickening is used to describe the process used to concentrate the solids content of the sludge from its initial value to a maximum solids content of about 8% in the case of primary or primary/humus sludge or about 6% in the case of waste activated sludge.

In figure 43, the reduction in the volume of sludge against an increase in concentration may be seen. The effect of increasing the concentration of waste activated sludge from, say, 0.5% to say 2.5% decreases the volume to 20% of the original volume.

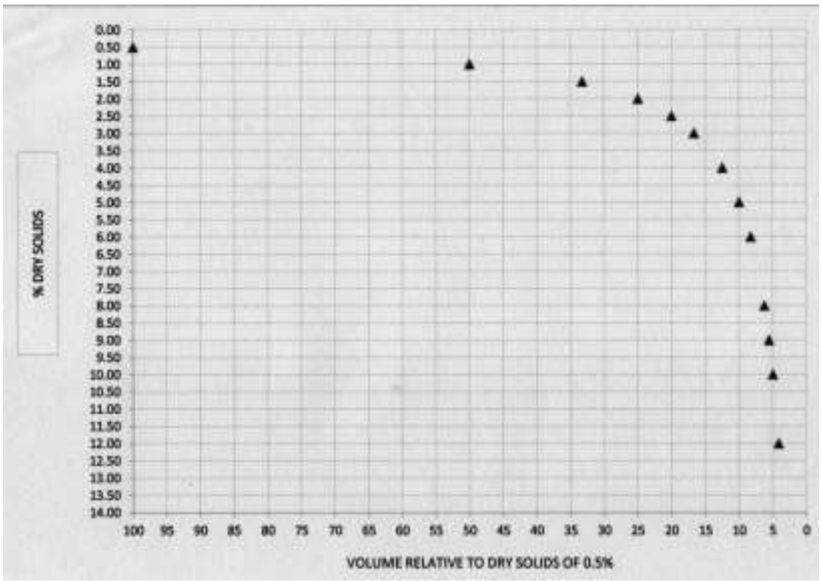


Figure 47 – SHOWING THE REDUCTION IN VOLUME WUITH INCREASE OF DRY SOLIDS CONTENT OF A SLUDGE.

There are two basic methods of thickening:

1. gravity thickening;
2. flotation thickening.

12.10 GRAVITY THICKENING.

This may be used for all types of sludges with varying effectiveness. The sludge that is withdrawn from primary or secondary sedimentation tanks may be thickened by resettling the sludge in a separate tank.

Earlier in section 7.8.1, it was seen that there are 4 separate categories of sedimentation. The two categories that dealt with low concentrations of suspended solid were dealt with in that section. The two categories that deal with the high concentration of suspended solids are covered here, i.e. zone settling and compression settling. See Figure 48.

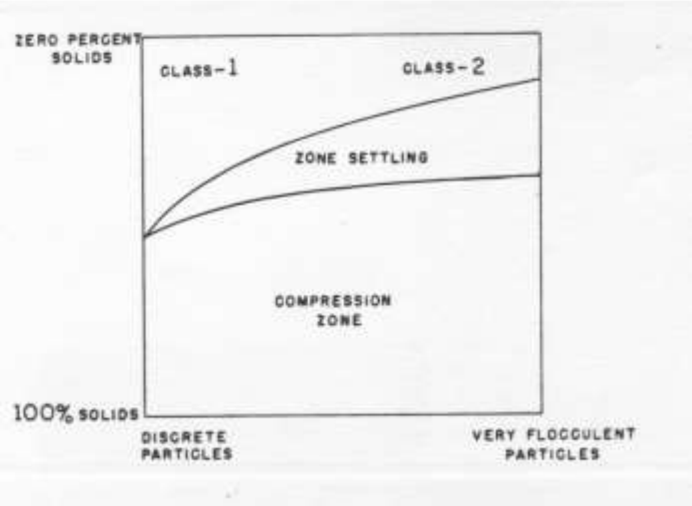


Figure 48 – SHOWING SETTLING AND COMPRESSION ZONES

When the particles of the solids that are settling are far enough from the boundaries of the container and from other particles, so that its fall is not affected by them, the process is called free settling. If the motion of the particles are impeded by other particles, which will happen when the particles are near each other even though they may not actually be colliding, the settling then becomes hindered. As the particles in settling displace liquid, the liquid assumes an upward velocity and this slows down the effective settling velocity of the solids. In zone settling, the water moving upwards tends to move in channels at a relatively high velocity. The particles will continue to settle but at a relatively low rate. As the sludge mass continues to settle, an inter-particle structure develops in the more concentrated lower layers and the settling rate slows to almost zero, and the compression settling stage is reached. Provided there is no liberation of gas or other biological activity, the ultimate concentration of the sludge will be reached.

The gravity thickener is constructed similarly to a primary sedimentation tank. In the past, the only sludge thickening practised was done in the primary sedimentation tank. However, unless very effective control is maintained over the desludging procedure, it is unlikely that the optimum solids content of the sludge will be obtained. As a sludge of relatively high concentration will have to be scraped to the centre, a floor having a steeper slope than a primary sedimentation tank may be necessary.

As the parts of the bridge underwater rotate they will encourage flocculation and the release of trapped gases. The movement of particles relative to each other will assist in the compaction by allowing water trapped between the sludge particles to escape.

As the quantity of water bound up in activated sludge is much greater than in primary and human sludges, it is not possible to get as good a consolidation. The typical upper limits for plain gravity thickening are about 8% in respect of primary only sludge, about 7% for a mixture of primary and humus, about 5% for a mixture of primary and waste activated and about 2% for waste activated only.

It is possible to improve on these values by using chemical dosing, but large doses are required and what is important - the total mass of sludge is increased. This chemical dosing must not be confused with chemical conditioning used prior to mechanical dewatering as described later.

It is usual to pass the effluent from the thickener back to the head of the works.

12.11 DISSOLVED AIR FLOTATION THICKENING OF WASTE ACTIVATED SLUDGE.

Waste activated sludge does not compact well when being thickened using gravity. In addition, the sludge is generally more active biologically and will release gases on storage, particularly carbon dioxide and maybe nitrogen as a result of denitrification. This gas generation tends to cause the sludge to become less dense. This can reduce the settling rate and if the gas evolution increases; particles of sludge may rise.

In order to get maximum compaction in activated sludge, the sludge mass must be kept aerobic or at least anoxic; it must not be allowed to become anaerobic. If waste activated sludge is thickened by gravity, and particularly in a mixture with primary sludge it is often difficult to prevent it becoming anaerobic.

In dissolved air flotation, air is added at pressures in excess of atmospheric pressure (usually at 400 to 700 kPa) to either the incoming sludge stream or to a separate liquid. The latter may be taken from the overflow of the tank. When the pressure is suddenly reduced and turbulence is created, the air in excess of that required for saturation at atmospheric pressure leaves the solution (precipitates) as very small bubbles of 50 to 100 µm diameter. The bubbles adhere to the sludge or become enmeshed in the solids matrix. Since the average density of the solids-air aggregate is less than that of water, the agglomerate floats to the surface. The floated solids build up to a depth of several centimetres at the water surface. A part of the float will project above the normal water surface. Water will drain from the upper level of the float and this will help to increase the solids content of the float. The float should be removed continuously by a skimmer. A typical process flow diagram is shown as figure 49. The dissolved air flotation unit may be round, square or rectangular.

The liquid that is to be pressurized to allow the air to dissolve may be the feed liquid or a recycled portion of the clarified liquid. If the feed liquid is to be pressurized, it may be necessary to treat only a portion of the feed.

The liquid to be pressurized may be passed through a plain pressure vessel with reaeration pump or the liquid may be passed through a packed column. The latter has the advantage of being very much smaller, but has the disadvantage that only clear liquid may be passed through the column; otherwise the column packing may become blocked.

The solids content of the float produced will be about 2 - 5% without chemical addition and 4 - 8% with addition of polyelectrolyte when treating waste activated sludge. Provided the quantity of air used is sufficient and the retention systems are suitable virtually all the sludge should rise to the surface and be removed. The overflow liquid will usually contain 20 - 200 mg/L of suspended solids.

Any sand etc. present in the feed sludge will settle to the bottom of the tank and may be removed separately from the floated sludge.

Among the other advantages of dissolved air flotation over gravity thickening are:

1. the better solids - liquid separation than with gravity thickener;
2. a higher solids content than with a gravity thickener;
3. requires less space;
4. reduced chance of odour problems;
5. grease is removed with the float;
6. any sand or other heavy material in the sludge will settle to the bottom of the tank and can be removed separately.

Among the disadvantages of dissolved air flotation are:

1. higher operating costs;
2. very little sludge storage capacity.

At this stage, the solids content of the sludge will be between 4 % and 8%, depending on the nature of the sludge. In most cases, the sludge will require further treatment to prevent it becoming smelly and difficult to handle. In other words, the sludge must be stabilized.

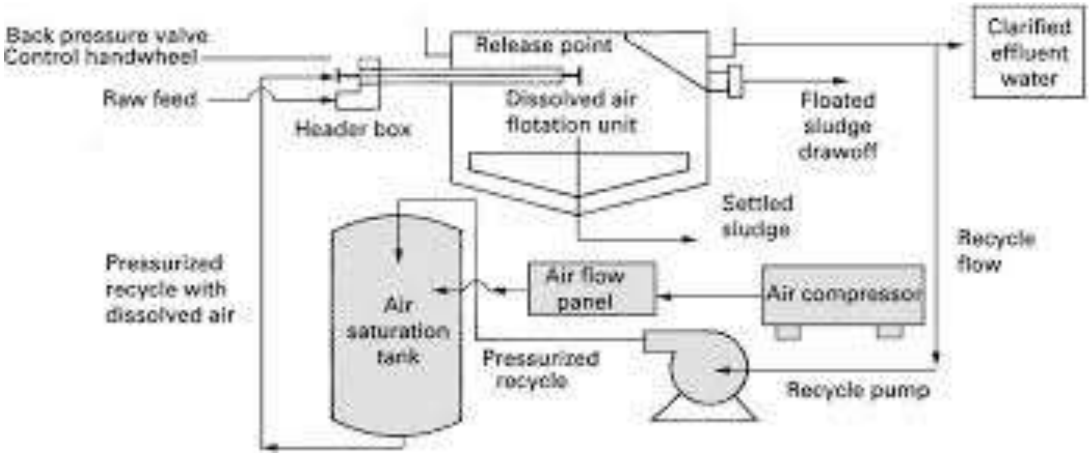


Figure 49 – A TYPICAL PROCESS FLOW DIAGRAM FOR DISSOLVED AIR FLOTATION THICKENING OF WASTE ACTIVATED SLUDGE.

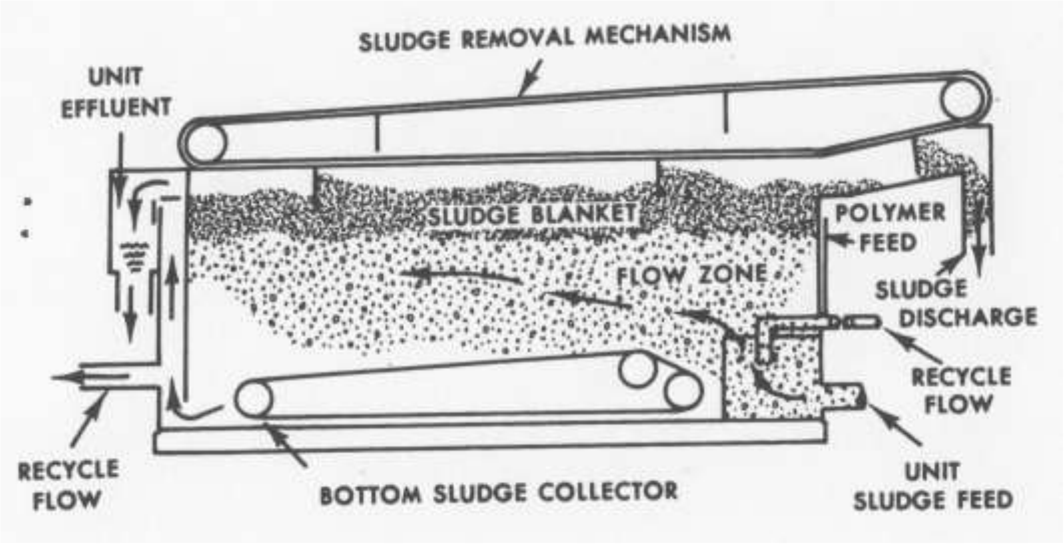


Figure 50 – A TYPICAL LAYOUT OF A DISSOLVED AIR FLOTATION UNIT.

WASTEWATER TREATMENT.

PART 13

SLUDGE TREATMENT - STABILIZATION PROCESSES.

13.1 INTRODUCTION.

The principal purposes of stabilization are to make the treated sludge less odourous and putrescible and to reduce the pathogenic organism content. Some procedures used to accomplish these objectives can also result in other basic changes in the sludge. The selection of a particular method hinges primarily on the final disposal procedure to be used. For example, if the sludge is to be dewatered and incinerated, the stabilization stage may be omitted. Most stabilization methods, except high lime treatment, result in a decrease in the mass of sludge being treated.

The stabilization processes that will be covered are:

1. anaerobic digestion;
2. aerobic digestion;
3. lime stabilization;
4. heat treatment;
5. composting.

13.2 ANAEROBIC DIGESTION.

13.2.1 Introduction.

Anaerobic digestion is the biological degradation of complex organic substances in the absence of free oxygen. During these reactions, energy is released and much of the organic matter is converted to methane, carbon dioxide and water. Since the energy value of the remaining solids is relatively small, the sludge is rendered stable.

Anaerobic digestion is among the oldest forms of biological wastewater treatment. It was first used a century ago to reduce both the quantity and odour of sewage sludges. Originally, anaerobic digestion was carried out in the same tank as sedimentation, but the two-story tanks developed in England by Travis and in Germany by Imhoff began a trend toward separating the two processes. Separate sludge digestion tanks came into use in the first decades of this century. At first, these were little more than simple holding tanks, but they provided the opportunity to control environmental conditions during anaerobic digestion and, thereby, improve process performance. With the development of digester heating and, subsequently, mixing, anaerobic digestion became the most common method of stabilizing sludge.

As both industrial waste loads and the general degree of wastewater treatment increased, the sludges generated by treatment plants became more varied and complex. Digester systems failed because their design and operation were empirically developed under simpler conditions. As a result, anaerobic sludge digestion fell into disfavour. However, interest in anaerobic digestion of dilute wastes stimulated a new wave of research into the process. The resulting development of steady state models in the 1960s, dynamic models in the 1970s, and increasing research into the basic biochemical processes, led to significant improvements in both reliability and performance of anaerobic digesters.

A 1977 survey of 98 municipal wastewater treatment plants in the United States found that 73 used anaerobic digestion to stabilize and reduce the volume of sludge. Because of emphasis on energy conservation and recovery and environmental pressure to use wastewater sludges on land, it is expected that anaerobic digestion will continue to play a major role in municipal sludge processing.

A wide variety of sludges from municipal wastewater treatment works can be stabilized through anaerobic digestion. Chemical Sludges have been successfully digested anaerobically, although in several cases, the volatile solids reduction and gas production have been low, compared with municipal wastewater sludges. It is likely though that this is as a result of reduced biodegradability rather than inhibitory effects. Waste activated sludge alone generally does not digest well anaerobically.

13.2.2 Advantages and Disadvantages of the Process.

Anaerobic digestion offers several advantages over other methods of sludge stabilization:

1. methane is produced, which is a usable source of energy. In most cases enough gas is produced to provide energy for heating and mixing the contents of the digester with any excess being used to heat buildings, running engines or for generating electricity;

2. a reduction in the sludge mass occurs as a result of the conversion of organic matter to methane, carbon dioxide and water. Usually 25 to 45% of the feed sludge solids are destroyed during anaerobic digestion. The actual loss being dependent on the type of sludge and the degree of stabilization used;

3. The treated sludge may be used as a soil conditioner. The anaerobically digested sludge may be used to improve the quality of soils. This will be covered in greater detail later in the disposal of sludges;

4. A large proportion of the pathogenic organisms present in the feed sludge will die off during the relatively long retention times used in anaerobic digestion.

Among the disadvantages of the process are:

1. The very large, closed digestion tanks are expensive to construct, in addition expensive heating and mixing systems are required;

2. The process is prone to upsets due to the sensitivity of the microorganisms to small changes in their environment;

3. The liquid fraction from the treatment process has high chemical oxygen demand, nitrogen and phosphorus contents. This liquid fraction may result from the withdrawal of supernatant from the digesters or from subsequent dewatering processes.

13.2.3 The Microbiology of the Anaerobic Digestion Process.

Anaerobic digestion involves several successive fermentations carried out by a mixed culture of microorganisms. This web of interactions compromises two general degradation phases: acid formation and methane production. Figure 51 shows, in simplified form, the reactions involved in anaerobic digestion.

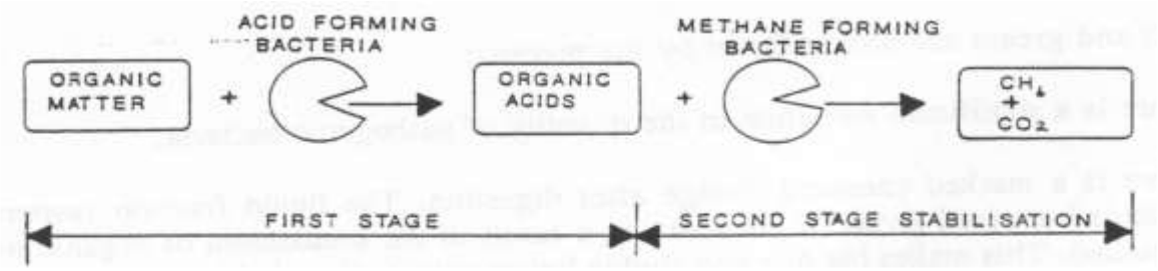


Figure 51 – A SIMPLIFIED ILLUSTRATION OF THE DECOMPOSITION OF ORGANIC MATTER INTO METHANE AND CARBON DIOXIDE.

In the first phase of digestion, facultative bacteria convert complex organic substrates to short-chain organic acids - primarily acetic, propionic, and butyric acids. These volatile organic acids tend to reduce the pH, although alkaline buffering materials are also produced. Organic matter is converted into a form suitable for breakdown by the second group of bacteria.

In the second phase, strictly anaerobic bacteria (called methanogens), convert the volatile acids to methane (CH_4), carbon dioxide (CO_2) and other trace gases. There are several groups of methanogenic bacteria each with specific substrate requirements that work together to reduce complex wastes such as wastewater sludge. Tracer studies indicate that there are two major pathways of methane formation:

1. The cleavage of acetic acid to form methane and carbon dioxide;

$$CH_3COOH \rightarrow CH_4 + CO_2$$

2. the reduction of carbon dioxide by hydrogen gas or formic acid (HCOOH) produced by other bacteria, to form methane.

$$O_2 + 4H_2 \rightarrow CH_4 + 2H_2O$$

When an anaerobic digester is working properly, the two phases of degradation are in dynamic equilibrium; that is, the volatile organic acids are converted to methane at the same rate that they are formed from the more complex organic molecules. As a result, volatile acid levels are low in a working digester. However, methane formers are inherently slow-growing, with doubling times measured in days. In addition, methanogenic bacteria can be adversely affected by even small fluctuations in pH, substrate concentrations, and temperature. In contrast, the acid formers can function over a wide range of environmental conditions and have doubling times normally measured in hours.

As a result, when an anaerobic digester is stressed by shock loads, temperature fluctuations, or an inhibitory material, methane bacteria activity begins to lag behind that of the acid formers. When this happens, organic acids cannot be converted to methane as rapidly as they form. Once the balance is upset, intermediate organic acids accumulate and the pH drops. As a result the methanogens are further inhibited, and the process eventually fails unless corrective action is taken.

The anaerobic process is essentially controlled by the methane bacteria because of their slow growth rate and sensitivity to environmental change.

Therefore, all successful designs must be based around the special limiting characteristics of these microorganisms.

13.2.4 The Effect of Temperature on Anaerobic Digestion.

There are 3 main temperature ranges in which anaerobic digestion may be operated. These are:

1.	cryophilic range	10 - 20ºC;
2.	mesophilic range	30 - 40ºC;
3.	thermophilic range	45 - 60ºC.

As will be seen later in the section on loading rates, the rate of digestion is the slowest in the cryophilic range and faster at the higher temperatures. It will be seen in Figure 53 that there is an optimum temperature within the ranges given.

As the anaerobic organisms, particularly the methanogens, are easily inhibited by even small changes in temperature, it is important that the temperature be kept as constant as possible. The temperature range should not be more than about 3ºC.

13.2.5 The Effects of Solids Retention Time on Anaerobic Digestion.

In order for the anaerobic digestion process to continue operating satisfactorily it is important that the bacteria have enough time to reproduce so that they can:

1. replace cells lost with the withdrawn sludge;

2. adjust their population size to follow fluctuations in organic loading.

In this respect a parameter called the solids retention time is used, this is similar to the term sludge age as used in the activated sludge process. Often the term solids retention time is used in the latter process as well. In digesters where no recycle is used, the solids retention time is equal to the hydraulic retention time and is calculated by dividing the volume of the digester by the volume withdrawn each day. If a recycle of concentrated sludge is used (as in the activated sludge), the SRT increases relative to the HRT.

If the SRT falls below a critical value, the process will fail because the bacteria are being lost at a rate greater than they can reproduce. This is one of the reasons for ensuring that excessive quantities of water are not added to the digester with the feed sludge. This is apart from the extra heat needed for the excess water.

Typical Solids Retention Times for the various temperature ranges are shown in table 9.

TABLE 9 – TYPICAL SOLIDS RENTION TIME OF DIFFERENT OPERATING REGIMES.

OPERATING REGIME	**TYPICAL SRT**
Cryophilic	30 to 60 days
Mesophilic	15 to 25 days
Thermophilic	5 to 12 days

As the SRT increases, the volatile solids reduction levels off as seen in figure 52.

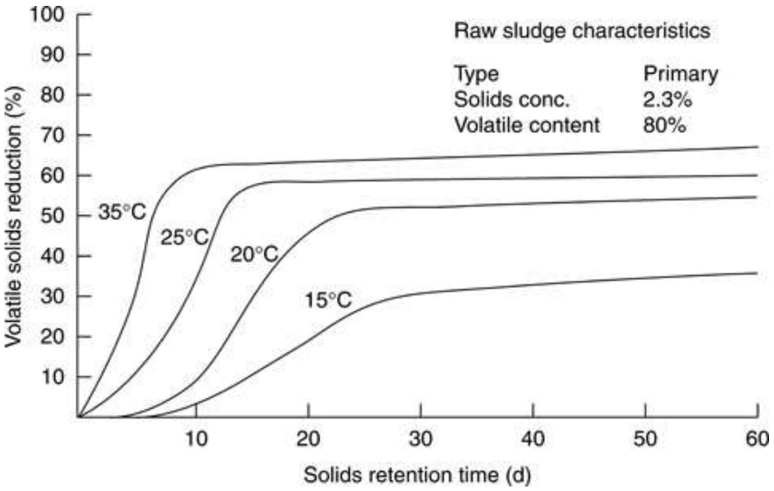


Figure 52 – VOLATILE SOLIDS REDUCTION vs SOLIDS RETENTION TIME.

13.2.6 The Effect of Loading Rate on Anaerobic Digestion.

There are two main ranges of loading rates used in anaerobic digestion; the so called low rate and high rate systems.

In the low rate system, the contents of the digester are neither heated nor mixed. This system would operate in the cryophilic temperature range. The SRT would be 30 - 40 days. The loading rate would be in the range 0.4 to 1.6 kg volatile suspended solids per cubic metre - day. The actual retention time required would be dependent on the ambient temperature as well as the "degree" of digestion required i.e. the volatile solids reduction needed. It was seen in figure 52, that there is little advantage in going beyond about 30 days when the sludge temperature is 15ºC. However, the volatile solids reduction is only about 35% at this point.

Although the volatile solids content of the raw sludge is usually about 80%, generally only about 70% of the volatile solids in typical municipal wastewater sludge is readily biodegradable solids are reduced.

In figure 52, it was seen that at higher operating temperatures, such as 35º, about 60% reduction in volatile solids may be obtained in only 12 days SRT. When a digester is operated at this temperature and SRT, the unit becomes known as the high rate process.

Typical loading values are 1.6 to 6.4 kg volatile suspended solids per cubic metre-day, with a mean value of about 4. The SRT is usually in the range 15 to 25 days. It is seen that the size of the high rate digester would be very much smaller than the size of a low rate digester used to handle the same mass or volume of sludge. It will be necessary to mix the contents of the digester for the reasons stated below.

Frequently a high rate digester is coupled in series with a second digester. Traditionally, this secondary digester is unmixed and unheated. Its main function has been to allow gravity concentration of the digested sludge and the decanting of supernatant liquor. This has the advantage of reducing the volume of sludge to be further handled. Very little volatile solids reduction and gas production takes place in the second stage.

Unfortunately, many secondary digesters have performed poorly as thickeners by producing little increase in the solids content of the sludge while producing a high strength supernatant. The basic cause is that usually anaerobically digested sludges do not settle readily. If it is desired to ensure that the sludge is always well digested, and that the maximum gas yield is obtained, it may be better to heat and mix the contents of the secondary digester so that the system is operated as though there are only primary digesters.

13.2.7 The Effects of Mixing and Pre-thickening on Anaerobic Digestion.

The sludge in high rate digesters is mixed continuously to create a homogeneous environment throughout the reactor. When stratification is prevented, the entire digester volume is available for active decomposition, thereby increasing the effective SRT. Also, what is most important, mixing brings the raw sludge into contact with the micro-organisms and evenly distributes metabolic waste products and any toxic substances present in the system.

It has been stated earlier that one of the advantages of thickening sludge before digestion was to decrease the volume of sludge to be handled. This is most important when heated digesters are used. If excessive quantities of water are added to the digester (in the form of a dilute sludge), there may be insufficient gas to provide the energy to heat the sludge to the desired operating temperature. This can result in the temperature of the contents of the digester dropping, this will result in less gas being produced and can lead to process failure.

Going to the other extreme by over-thickening the sludge can also cause problems. Amongst these are:

1. Good mixing becomes difficult to maintain. The solids concentration in the digester affects the viscosity of the sludge which in turn affects the mixing. Some Process Controllers have reported difficulty in mixing when the solids content in the digester exceeded 6%. Because of the reduction of volatile solids occurring during digestion, the solids concentration within the digester is less than the feed solids concentration. Therefore, feed solids concentrations may reach 8 - 9% before mixing is impaired;

2. Chemical concentrations can reach levels that can inhibit microbial activity. A highly thickened feed sludge means that the contents of the digester will be very concentrated. Compounds entering the digester, such as salts and heavy metals, and end products of digestion, such as volatile acids and ammonium salts, may reach concentrations toxic to the bacteria in the digester. It is possible that poor mixing could allow toxic concentrations to occur in certain areas in the digester.

One of the problems of anaerobic digestion is that a very strong liquid fraction is produced. By pre-thickening, the volume of supernatant is decreased, but as has been seen above there is a limit above which one cannot go.

13.2.8 Feeding of a Digester.

Ideally digesters should be fed at a uniform rate but this is usually not the case. Sludge is usually drawn from the primary sedimentation tank on an intermittent basis and if pre-thickened is usually drawn at regular intervals. To feed sludge continuously would require either the automation of upstream components or an increase in the operating staff.

By feeding a digester as frequently as possible, this helps to maintain constant conditions within the reactor. It also helps to equalize the gas production thus reducing wastage when the excess gas cannot be stored.

13.2.9 Volatile Solids Reduction.

Solids reduction is one of the main objectives of anaerobic digestion. It not only makes the sludge more stable but also reduces the amount of solids for final disposal. As only the volatile portion of the sludge solids is reduced, the parameter used as a common measure of digester performance is the percentage of volatile solids reduced. The volatile solids reduction in anaerobic digestion is usually in the range 35 to 60%. The degree of volatile solids reduction achieved in any particular application depends on both the character of the sludge and the operating parameters of the digestion system i.e. the SRT and the temperature.

The character of the sludge determines the upper limit for the volatile solids reduction since not all the volatile solids can be converted by the anaerobic bacteria. The percentage of the volatile solids that is non-biodegradable is about 40% for primary sludge, about 25% for waste activated sludge after primary sedimentation and about 30% for waste activated sludge without primary sedimentation. On the average thus, only between 60 and 80% of the volatile solids present in wastewater sludge are readily biodegradable. In this case, one is referring to the 15 to 25 days retention time in digesters; if the sludge is held for several months in a lagoon, then the percentage of biodegradability will usually increase.

The most important operating parameters affecting volatile solids reduction are the solids retention time and the digestion temperature. The volatile solids reduction vs solids retention time was shown in figure 52.

13.2.10 Gas Production.

Possibly the most important advantage of anaerobic digestion over other methods of sludge stabilization is that it produces a gas which can be burnt to provide energy. Besides its use as a fuel, it may also be used as a chemical feedstock to make methanol and cyanide.

The generation of digester gas is as a direct result of the destruction solids. The microbiology and biochemistry of this conversion was covered in section 13.2.3. Because of this close relation between gas production and solids reduction, gas production is best expressed in terms of the volume of gas produced per unit of solids destroyed. This specific gas production is usually expressed in cubic metres of gas per kilogram of volatile solids destroyed. The specific gas production and the percentage of that gas that is methane is shown below in table 10, for various compounds in municipal sludge.

TABLE 10 – GAS QUANTITY FROM ANAEROBIC DIGESTION OF VARIOUS MATERIALS.º

MATERIAL	m^3 gas/kg VS DESTROYED	% METHANE
Fats	1.1 - 1.5	62 - 72
Scum	0.8 - 1.0	70 - 75
Grease	1.1	65 - 70
Crude Fibres	0.8	45 – 50
Proteins	0.75	70 - 75

Figure 53 shows how the gas production is affected by temperature. Conversion of volatile solids is most efficient at about 35ºC (the mesophilic optimum temperature) and at about 54ºC (the thermophilic optimum temperature). The solids retention time has no effect on the specific gas production, but does of course have an effect on the total gas production. It may also be seen in figure 53 that the specific gas production falls off quite steeply on either side of the optimum temperature. Hence the need to keep the temperature in the digester as constant as possible and as close to the optimum temperature as possible.

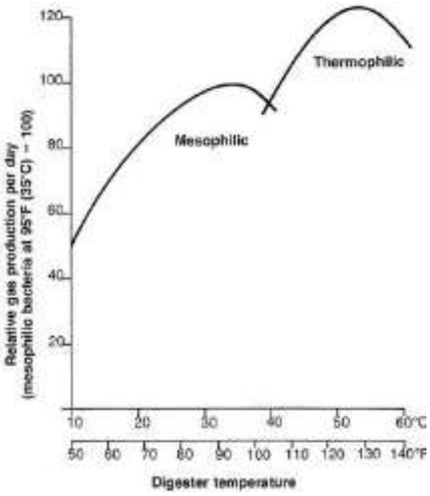


Figure 53 – RELATIVE RATE OF GAS PRODUCTION WITH TEMPERATURE.

TABLE 11 - TYPICAL COMPOSITION OF DIGESTER GAS

PARAMETER	**TYPICAL VALUE RANGE**
Methane content	60 - 75% by volume
Carbon Dioxide content	25 - 40% by volume
Hydrogen content	0.5 - 2.5 % by volume
Nitrogen content	1 - 5% by volume
Hydrogen Sulphide content	0.01 - 1.0% by volume
Heat Value	18 - 26 MJ/m^3

By comparison liquefied natural gas (LNG) has a heat value of about 30 MJ/m^3, while liquefied petroleum gas (LPG) has a heat value of about 46 MJ/m^3.

13.2.11 Supernatant Quality.

The supernatant from an anaerobic digestion system can contain high concentrations of organic material, both dissolved and suspended. Here the term supernatant refers to either liquid withdrawn from the upper layer of an unmixed digester or the liquid fraction from one of the dewatering processes that will be covered later.

The returning of supernatant to the usual treatment processes imposes an additional load that must be allowed for when determining the treatment capacity of the works. Typical values of parameters are detailed below in table 12:

TABLE 12 - TYPICAL SUPERNATANT QUALITY

PARAMETER	**TYPICAL CONCENTRATION**
Chemical Oxygen Demand	1 000 – 4 000 mg/L
Total Phosphorus as P	60 – 150 mg/L
Ammonia Nitrogen as N	250 – 850 mg/L
Volatile Acids as Acetic Acid	200 – 300 mg/L
Suspended Solids	100 – 1 000 mg/L

As the organic materials in the supernatant are in a chemically reduced state, they will impose a large and often sudden oxygen demand. For this reason, supernatant should be returned over as long a period as possible each day. During the normal biological process, phosphorus is removed from the water by the creation of new cell material, but during anaerobic digestion cell material is broken up and the phosphorus is released. This release of phosphorus will reduce the overall removal and may necessitate the use of chemical treatment to remove the phosphorus to an acceptable level.

The suspended solids present in the supernatant will tend to be the finer particles. These particles will be the first to be lost over the secondary sedimentation tank weir and will reduce the quality of the effluent

13.2 12 Engineering Aspects 1 - Tank Design.

Anaerobic digesters may be cylindrical, rectangular or egg-shaped. These shapes are seen in figure 54.

The cylindrical shape is the most common, with diameters of 6 to 4 m and side wall depths of 6 to 12 m. The floor of the cylindrical digester is usually conical with a slope of at least 1:6. The sludge is withdrawn from a low point in the centre of the tank.

Rectangular digesters are cheaper to build but mixing is more difficult and dead spots tend to occur in the corners.

The egg shaped digester is the easiest to mix and discourages grit accumulation, but is very much more expensive to build.

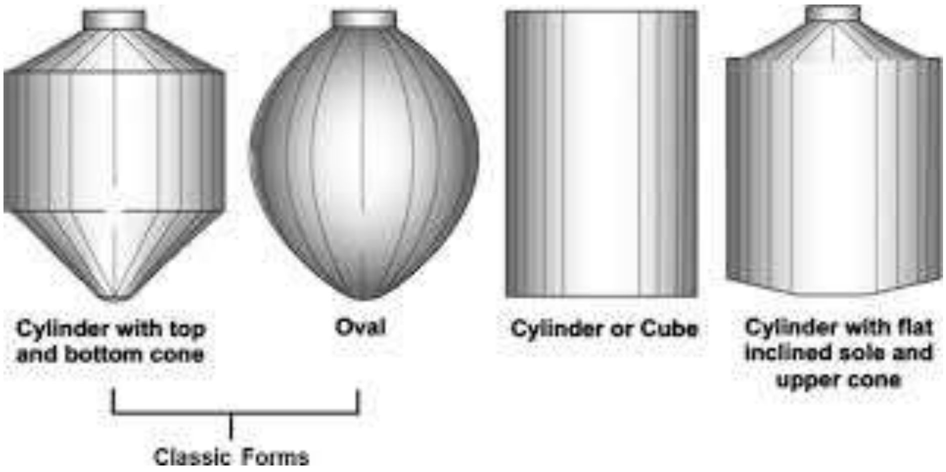


Figure 54 – VARIOUS SHAPES OF ANAEROBIC DIGESTER.

13.2.13 Engineering Aspects 2- Heating.

The need to heat digesters has been covered earlier. Maintenance of the operating temperature within a range of 3º C and preferably within a range of 1º C improves the process stability by preventing thermal shock. There are four methods used for heating the contents of a digester:

1. internal heat exchanger. Here the heat exchanger coils are installed inside the digester and hot water is passed through the coils. The heat passes from the water through the pipe wall into the sludge. These have the disadvantage that they cannot be inspected, repaired or cleaned;

2. external heat exchanger. Here the sludge is withdrawn from the digester, passed through a heat exchanger and then returned to the digester. Being outside the digester, the unit may be readily inspected and cleaned in order to maintain the heat transfer efficiency;

3. direct steam injection. Here live steam is injected directly into the sludge. Although this is the simplest process it has the disadvantage that fresh water must be added to the boiler continuously. The condensing steam will dilute the sludge and those areas of sludge in direct contact with the steam undergo a thermal shock.

The hot water or steam used to heat digesters is most commonly generated in a boiler fuelled by sludge gas. Up to 75% of the heat value in the sludge gas can be recovered in the boiler. It will be necessary to have an auxiliary heat source to maintain heating during periods of low gas production or high heat demand. This is necessary as it has been seen that only a slight drop in temperature in the digester leads to a marked fall off in gas production which in turn means that less gas is available and so the temperature drops further and the process failure becomes imminent. Auxiliary fuel sources include natural gas, diesel, furnace, oil and coal.

If the digester gas is used to drive an internal combustion engine, then heat may be extracted from the engine cooling water and from the exhaust gases. Some experiments have been done using solar energy; the main disadvantages of this system are the high capital costs and the need for an auxiliary heat source for nights and during winter.

As seen earlier in 12.9, a small drop in the dry solids content of the feed sludge has a significant increase in the volume being fed to the digester. This in turn, significantly increases the amount of energy required to heat the feed to the temperature of the body of the digester.

13.2.14 Engineering Aspects 3 – Mixing.

The need for effective mixing has been detailed above. There are 3 methods used to mix the contents of a digester:

1. external pumped circulation. This is the simplest method but large flow rates are needed to ensure adequate mixing. One advantage of this method is that it may be combined with the heating process, and with the raw sludge feeding system. Input energy requirements seem to be in the range 5 to 8 $W/m^{3;}$

2. internal mechanical mixing. Here a propeller or turbine mixer is used. The mixer may be installed through the wall, or more commonly through the roof. Problems have occurred with rags, etc., becoming entangled around the mixer, obstructing the mixing action and even causing failure of the equipment. Required power inputs appear to be of the same order as in point 1 above;

3. internal gas mixing. Several variations of gas mixing are used, some with draft tubes and some without. In all cases gas is withdrawn from the digester, compressed and injected. Required power inputs are of the same order as 1. and 2. above.

No mixing system has proved itself to be superior to another. In many cases poor mixing has been due to inadequate power inputs and in the case of gas mixing obstruction due to a deep bed of grit building up.

13.2.15 Engineering Aspects 4 - Gas Storage.

The rate of gas production is not constant and will at times exceed the normal gas usage rate and will at other times be insufficient to meet demands. Some form of gas storage is thus desirable.

The gas may be stored in separate gas holders at pressures between 2 and 4 kPa in low pressure units and at somewhat higher pressures in high pressure units. Some digesters have been fitted with floating roofs that move up and down in sympathy with excess gas production.

13.2.16 Operational Aspects 1 - pH Value.

As seen earlier, anaerobic digestion is a two-step process consisting of "acid-forming" and "methane-forming" steps. During the first step, the production of volatile acids reduces the alkalinity and the pH. The reduction is normally countered by destruction of volatile acids by methanogenic bacteria and the subsequent production of bicarbonate that increases the alkalinity.

Close pH control is necessary because methane-producing bacteria are extremely sensitive to slight changes in pH value. Research has shown that the optimum pH for methane-producing bacteria is in the range of 6.6 - 7.4 and that these bacteria are very sensitive to pH change. A 1970 study seems to indicate that the pH tolerance of methane producing bacteria is greater than previously thought. The bacteria are not necessarily killed by high and low pH levels; their growth is merely stopped. Because of the importance of these findings to system control, more research is needed to verify these results.

Several different acid-base chemical equilibria are related to pH value. In the anaerobic digestion process, the pH range of interest is 6.6 to 7.4, which makes the carbon dioxide-bicarbonate relationship the most important.

As figure 55 indicates, system pH is controlled by the CO_2 concentration of the gas phase and the bicarbonate alkalinity of the liquid phase. A digester with a given gas-phase CO_2 concentration and liquid-phase bicarbonate alkalinity can exist at only one pH value. If bicarbonate alkalinity is added to the digester and the proportion of CO_2 in the gas phase remains the same, digester pH must increase.

The pH increase is less important, however, than the effect on system buffering capacity, (that is, the system's ability to resist pH changes). If bicarbonate alkalinity is added, buffering capacity is increased, system pH is stabilized, and the system becomes less susceptible to upset.

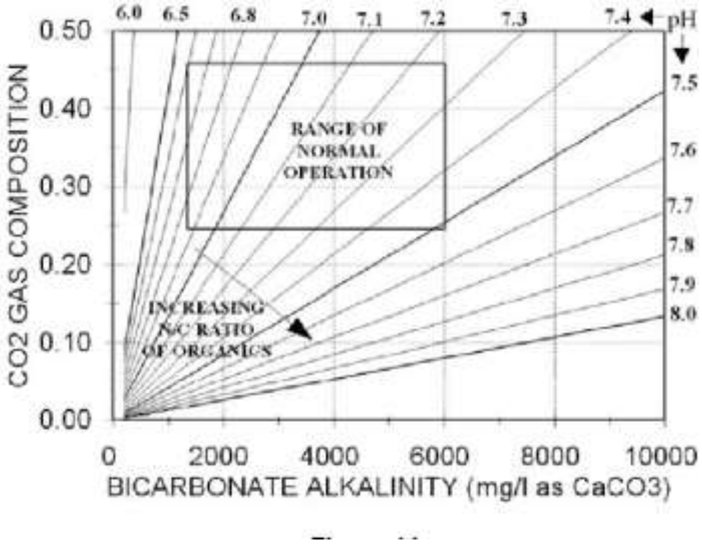


Figure 55 – SHOWING RELATIONSHIP BETWEEN GAS COMPOSITION AND ALKALINITY.

13.2.17 Operational Aspects 2 – Total and Bicarbonate Alkalinity.

The alkalinity is a measure of the content of alkaline material in the liquid i.e. alkaline with respect to pH 4.0 and is a measure of the buffer capacity or the ability to resist pH change.

Total alkalinity is determined by titration to pH 4.0.

Bicarbonate alkalinity can be calculated from total alkalinity by the following equation:

$$BA = TA - 0.71\,(VA)$$

where

BA = bicarbonate alkalinity as mg/l $CaCO_3$

TA = total alkalinity as mg/L $CaCO_3$ determined by titration to pH 4.0

VA = volatile acids measured as mg/L acetic acid

The factor of 0.71 is obtained by the multiplication of two factors: (0.83 and 0.5).

The factor 0.83 converts volatile acids as acetic acid to volatile acid alkalinity as $CaCO_3$.

The factor 0.85 is used because in a titration to pH 4.0; about 85 percent of the acetate has been converted to the acid form.

Bicarbonate alkalinity may be increased by adding sodium bicarbonate. Lime and sodium hydroxide can also be used but have the disadvantage of removing carbon dioxide from the gas phase and can cause a vacuum to develop in the system.

13.2.18 Operational Aspects 3 - Volatile Acids.

The level of volatile acids cannot be controlled. If the system is in balance, the volatile acid content should be in the range 2 000 to 7 000 mg/L as acetic acid. What is important is to maintain the ratio of volatile acids to total alkalinity in the range of 0.1 to 0.36.

An increase in volatile acid content may be balanced to some degree by increasing the total alkalinity.

13.2.19 Operational Aspects 4 - Toxicity Effects.

For any material to be biologically toxic it must be in solution. Toxicity is a relative term. Many substances may be beneficial at low doses and toxic at high doses. A good example is ammonia nitrogen as shown in table 13:

TABLE 13: IMPACT OF INCREASING AMMONIA CONCENTRATION ON THE PROCESS.

AMMONIA as N mg/L	EFFECT ON PROCESS
50 – 200	Beneficial effect
200 – 1 000	No adverse effects
1 000 – 3 000	Inhibitory above about pH 7.5
Above 3 000	Toxic

In many cases, the organisms present in the digester can become acclimatized to levels of potentially toxic materials that would poison an unacclimitized unit. Certain cations can increase the toxic effect of other cations. For example, the potentially toxic effects of ammonia can be aggravated by high calcium and magnesium levels while they are diminished by sodium. However, ammonia aggravates the toxic effects of high sodium levels.

Heavy metals are generally toxic, but under the conditions found in anaerobic digesters a large proportion of the metals is present as the carbonate, oxide or sulphide and are thus sparingly soluble. With heavy metals, prevention is better than cure and their entry into the sewer system should be strictly controlled to prevent problems occurring in the digester.

13.2.20 Operational Aspects 5 - Monitoring the Performance of Anaerobic Digesters.

The ultimate test of the digestion process is whether the final sludge is stable, non-putrescible and has a sufficiently reduced volatile solids content. For obvious reasons, one requires a method of monitoring the in-process performance of a digester system. Among the parameters that should be monitored are:

1. the gas production - a reduction in the volume gas produced under constant sludge loading conditions will give advance warning of possible process failure;

2. the carbon dioxide content of the gas - as an imbalance starts to occur; there will be an increase in the percentage of carbon dioxide as the methane producers become incapable of functioning. Under normal operation the carbon dioxide content of the gas should be 25 - 40% by volume;

3. the pH value - if used by itself this is not a good control procedure because it is a logarithmic function and is not very sensitive to large fluctuation in the bicarbonate alkalinity. For example, a change in bicarbonate alkalinity from 3 600 mg/L to 2 200 mg/L would change the pH value from 7.1 to 6.9 only. This is a small change. The other important disadvantage is that it does not provide adequate warning. A low pH value merely informs one that an upset has already occurred;
4. the bicarbonate alkalinity - by itself this analysis means little. Only when plotted as a function of time or used in conjunction with the volatile acid / bicarbonate ratio can impending operation problems be interpreted early enough to allow some sort of corrective procedure;
5. the volatile acids content - as above this analysis means nothing by itself;
6. the operating temperature - this is monitored to ensure that the physical conditions remain optimal for the digestion process;
7. the solids loading rate - this is monitored so that the digester is not becoming overloaded or that the solids retention time is not being reduced towards the critical value below which process failure will occur;
8. the volatile solids content of the sludge - this is to monitor the overall effectiveness of the process.

13.3 AEROBIC DIGESTION.

13.3.1 Introduction.

Aerobic digestion is the biochemical oxidative stabilization of wastewater sludge in tanks that are separate from the liquid process system. The aerobic digestion of municipal wastewater sludges is based on the principle that, when there is inadequate external substrate (food) available, the micro-organisms metabolize their own cellular mass. They require oxygen to perform this, so this is why the process is described as aerobic digestion.

In actual operation, aerobic digestion involves the direct oxidation of any biodegradable matter and the oxidation of microbial cellular matter by the organisms. These two steps are illustrated by the following reactions:

$$\text{organic matter} + O_2 \rightarrow \text{cellular material} + CO_2 + H_2O$$

$$\text{cellular material} + O_2 \rightarrow \text{digested sludge} + CO_2 + H_2O$$

The process described by the latter equation is referred to as endogenous respiration and is normally the predominant reaction in aerobic digestion.

A wide variety of sludges from municipal wastewater treatment works can be stabilized through aerobic digestion. Ordinarily there would be little purpose in aerobically digesting primary sludge in a works where secondary treatment stage is activated sludge, one would merely bypass the primary sedimentation tanks and pass the raw wastewater directly into the aeration tank. One could make a case for aerobically digesting the primary and humus sludge from a biological filtration works.

The main use for aerobic digestion is in the further treatment of waste activated sludge, particularly from works that are operated at a short sludge age, i.e. less than about l0 days.

13.3.2 Advantages and Disadvantages of the process.

Among the advantages are:

1. usually the capital costs for the structures and equipment is lower;

2. the aerobic system is generally easier to operate;

3. no nuisance odours are generated during the treatment process;

4. the supernatant is of a much higher quality;

5. pathogen reduction is usually better;

6. if the aerobic digester is operated in a nitrification/ denitrification mode, the effluent nitrogen content can be very low.

Among the disadvantages are:

1. the treated sludge concentration at about 1.5 to 2% solids is much lower than from an anaerobic digester, where about 4% solids is obtained. This means that for the same mass of sludge, the aerobically digested sludge will have a much larger volume.

2. if the aerobic digestion process is carried too far, a sludge with poor settling and dewatering characteristics is produced.

3 as no combustible methane is produced, there is a nett input of energy

4. In areas where the ambient temperature can be very low, the effectiveness of the process decreases during winter.

5. in areas where the water is poorly buffered, a significant drop in the pH can occur as a result of nitrification; however, if denitrification is practised some recovery of alkalinity will occur.

13.3.3 Microbiology of the Aerobic Digestion Process.

As the aerobic digestion process is essentially the activated sludge process operated at a long sludge age, the organisms found will be similar to those found in the activated sludge process.

Due to the long sludge age, usually greater than 25 days, protozoa and ciliates will be present in relatively large numbers. The various bacteria responsible for nitrification will also be present.

13.3.4 Effect of Temperature on Aerobic Digestion.

Since the majority of aerobic digesters are open tanks, the digester liquid temperatures are dependent on weather conditions and can fluctuate extensively. As with all biological systems, lower temperatures retard the process while higher temperatures speed it up.

A new process, upon which much research is directed, is the auto-heated thermophilic aerobic digestion process. Here the feed sludge is pre-thickened to more than 4% and then aerated. Among the advantages claimed are higher rates of volatile solids reduction, the production of a pasteurized sludge and a lower oxygen requirement since no nitrification will occur at the process temperature of about 60ºC.

Among the disadvantages of this process when compared with the conventional aerobic digestion process are the need to pre-thicken using dissolved air flotation or a centrifuge (gravity thickening will not yield 4% solids for waste activated sludge alone), the greater mixing requirements due to the higher solids content, the need to conserve heat and the relatively high ammonia content of the effluent since nitrification does not occur.

13.3.5 Effects of Solids Retention Time on Aerobic Digestion.

If waste activated sludge only is to be aerobically digested then since the correct micro-organisms are already present, the concept of limiting solids retention time is not of concern. The greater the solids retention time the greater the reduction in volatile solids content.

13.3.6 Effect of Loading Rate on Aerobic Digestion.

A higher loading rate will require a higher rate of oxygen input. The dissolved oxygen content of the liquid can be easily monitored and the oxygen input varied as necessary. Too low a dissolved oxygen content will reduce the rate of endogenous respiration, unless nitrate is present as an alternative oxygen source while too high an oxygen content will be wasteful. A suitable dissolved oxygen content is 1 - 2 mg/L.

13.3.7 Effects of Mixing and Pre-thickening on Aerobic Digestion.

It is important that effective mixing takes place to ensure contact between the dissolved oxygen and the micro-organisms. Usually when the overall oxygen input is enough to maintain the process, the mixing related to the oxygen transfer process is sufficient.

Pre-thickening is of little advantage except when the auto-heated thermophilic process is used. When the aerobic digestion system is operated with a secondary sedimentation tank and a sludge return, i.e. just like the usual activated sludge process, then thickening will occur naturally.

13.3.8 Feeding of an Aerobic Digester.

This is not nearly as critical as in anaerobic digestion. If the loading rate exceeds the oxygen input rate, anoxic conditions will occur and denitrification will result. If the period of anoxia becomes excessive, the reduction in volatile solids destruction will occur. If the situation is allowed to deteriorate, anaerobic conditions may occur, the sludge will become black and start to smell.

As before, feeding the digester as frequently as possible ensures the best possible process stability.

13.3.9 Volatile Solids Reduction.

The volatile solids reduction may reach 60%, but at 20ºC up to 40 days solids retention time may be required.

13.3.10 Gas Production.

The only gases produced are carbon dioxide and when denitrification is practised, also nitrogen. However, these are contaminated with air and as they have no fuel value, they are not collected.

13.3.11 Supernatant Quality.

The supernatant is generally of reasonably high quality and may be returned to the head of the works for re-processing.

Typical values of the parameters are detailed below in table 14:

TABLE 14 – TYPICAL EFFLUENT QUALITY FROM AEROBIC DIGESTER

PARAMETER	TYPICAL VALUE IN mg/L
Chemical Oxygen Demand	100 - 400
Ammonia as N (if no nitrification)	50 - 150
Ammonia as N (if full nitrification)	Less than 3
Nitrate as N (without denitrification)	30 - 100
Nitrate as N (with denitrification)	Less than 20
Suspended Solids	20 - 300
Phosphorus as P	20 - 100

Phosphorus may be precipitated by the addition of an iron salt before or during the digestion process.

13.3.12 Engineering Aspects.

The aerobic digester may be regarded as an activated sludge treatment works with sludge as a feed material.

13.3.13 Operational Aspects - Monitoring the Performance of Aerobic Digesters.

The ultimate test of the digestion process is whether the final sludge is stable, non-putrescible and has a sufficiently high volatile solids reduction. Among the parameters that should be monitored are:

1. the dissolved oxygen content - an insufficient dissolved oxygen content will prevent nitrification taking place and can inhibit the whole process;
2. the pH value - in poorly buffered waters inhibition of nitrification can occur if the pH is depressed too far. Denitrification will help to restore the balance;
3. the settleability of the sludge - a poorly settling sludge is usually an indication that all is not well;
4. the colour of the sludge - the sludge should be a rich chocolate brown;
5. the appearance of the sludge - the sludge should have a strong floc structure, which after being broken up by vigorous stirring should reform within a few minutes when aided by gentle stirring;
6. the odour of the sludge - the sludge should have a faint fresh earthy smell;
7. the volatile solids content of the sludge - this is to monitor the overall effectiveness of the process.

13.4 LIME STABILIZATION.

13.4.1 Introduction.

Originally lime was added to sludge to improve the dewaterability of the sludge. It was noticed that at fairly high doses of the order of 10% of the dry solids content there was significant odour and pathogen level reduction.

All types of municipal sludge can be treated using this method. The high lime dosing of sludge affects the chemical and physical characteristics of the sludge.

13.4.2 Advantages and Disadvantages of the Process.

Among the advantages of high lime treatment are:

1. the process may be used as a back-up for other processes as it can be easily started and stopped. It may be used to handle an additional load or may be used when existing equipment is being serviced or repaired;
2. when the sludge is added to soil, calcium is added as well;
3. pathogen die-off and virus inactivation is usually superior to either of the digestion processes;
4. the short contact time required. The contact period need only be a few hours compared with the many days needed for the digestion processes;
5. the capital costs of the civil equipment is very much less, both due to the reduced size and also due to the simplicity of the process;

6. as the process does not rely on biological reaction, the process is not prone to inhibition by materials toxic to the organisms.

Among the disadvantages are:

1. the mass of sludge is increased during processing, in contrast with the other processes;

2. the sludge is not permanently stabilized by this process. If the sludge was disposed of in an unsatisfactory manner, odour problems could re-occur if the pH value were to drop.

13.4.3 The Effect of pH Value on Lime Stabilization.

The primary objectives of lime stabilization are to inhibit bacterial decomposition and to inactivate pathogenic organisms. The effective factor in lime stabilization is evidently the pH value attained and not merely the lime dose used. The design objective is to maintain the pH value above 12 for about 2 hours and also to provide sufficient alkalinity to ensure that the pH does not drop below 11 for several days, allowing sufficient time for disposal.

13.4.4 The Effect of Contact Time on Lime Stabilization.

The required contact time was detailed above in 15.4.3. Usually the pH is raised to 12.5 for 30 minutes; this keeps the pH above 12 for the required two hours.

13.4.5 The Lime Dose required for Lime Stabilization.

An indication of the lime dose required to keep the pH above 12 for 2 hours and above 11 for 14 days is indicated in the table 15:

TABLE 15 – TYPICAL LIME DOSAGE FOR DIFFERENT SLUDGES.

SLUDGE TYPE	**TYPICAL LIME DOSE** kg per kg dry solids
Primary Sludge	0.10 – 0.15
Mixed Primary and Waste Activated	0.15 – 0.20
Waste Activated	0.20 – 0.50
Septic Tank	0.10 – 0.40
Anaerobically Digested Primary and Waste Activated	0.15 – 0.25

13.5 HEAT TREATMENT.

In this section, heat treatment is taken to mean treatment at a temperature of above 100ºC

13.5.1 Introduction.

Heat treatment at temperatures of above 100ºC has also been called thermal conditioning, as one of its main uses is to condition the sludge so that it may be easily dewatered to a relatively high solids content i.e. above about 30%. There are two basic processes; the one is operated with the addition of air and the other without. In both these processes, temperatures of 150 to 260ºC and pressures of 10 bar to 30 bar (1 MPa to 3 MPa) are attained for about 30 minutes. During this time significant changes in the nature and composition of the wastewater sludges result.

The effect of heat treatment is to break down the sludge into water and residual solids. Wastewater sludges are essentially cellular material. These cells contain intracellular gel and extra-cellular zoogleal slime containing carbohydrates and protein. The heat treatment process breaks the cells and releases the protoplasm. The protein and zoogleal slime is broken down into simpler compounds. The solid material left is mineral matter and cell wall debris.

13.5.1 Heat Treatment without the Addition of Air.

In this process, sludge is macerated and pumped through a heat exchanger into a reactor. Steam is added to heat the sludge to the operating temperature. On leaving the reactor, the sludge passes through another heat exchanger, and a pressure control valve into a solids separation unit.

The Porteous Process is an example of this method. A flow sheet is shown in figure 56.

There is little actual stabilization of the sludge in this process. The treated sludge usually smells although it does dewater easily. One disadvantage of the process is that a substantial quantity of material is solubilized. After processing, the sludge is dewatered. The liquid discharge from the sludge dewatering unit is very strong as may be seen in table 16 below:

TABLE 16 – TYPICAL FILTRATE COMPOSITION AFTER DEWATERING PROCESS

PARAMETER	TYPICAL VALUES
Biochemical Oxygen Demand	1 600 – 12 000 mg/L
Chemical Oxygen Demand	2 500 – 25 000 mg/L
Total Nitrogen as N	700 – 1 700 mg/L
Total Phosphorus as P	70 to 100 mg/L
Colour on Hazen Scale	2 000 – 3 000 Units

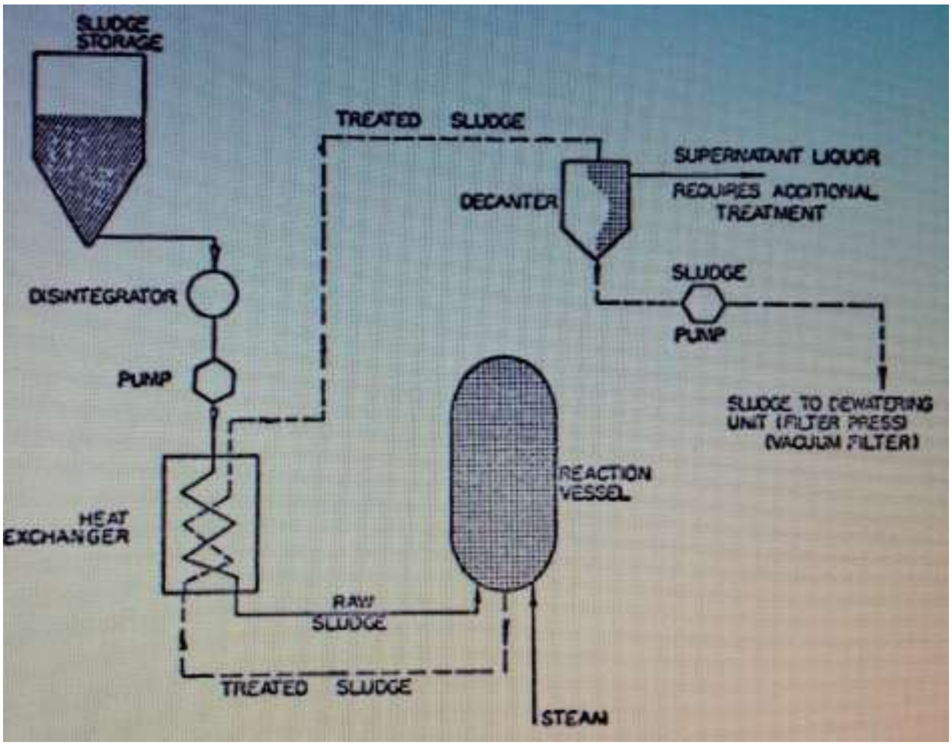


Figure 56 – FLOW SHEET OF THE PORTEOUS PROCESS.

If this liquid fraction is returned to the head of the works as is usual, this can add a significant extra load onto the works. It appears that about 25% of the COD is non-biodegradable within 30 days in the conventional treatment processes.

13.5.2 Heat Treatment with the Addition of Air (Wet Air Oxidation).

With the addition of air, some oxidation of organic materials takes place in the reactor, resulting in the production of heat. Under certain conditions, the process can become auto-thermic once established. By comparison, in the process without air addition, a continuous input of heat is required.

The Zimpro process is an example of this method. A flow sheet is shown in figure 56. In this case a vacuum filter is used – other types of dewatering equipment may be used. In this process, a strong supernatant is also produced. This may be seen in the following table where typical values of certain parameters of the liquid fraction after the dewatering process, are given in table 17:

TABLE 17 – TYPICAL FILTRATE COMPOSITION AFTER DEWATERING PROCESS.

PARAMETER	TYPICAL VALUES
Biochemical Oxygen Demand	5 000 – 15 000 mg/L
Chemical Oxygen Demand	10 000 – 30 000 mg/L
Total Nitrogen as N	650 – 1 000 mg/L
Total Phosphorus as P	150 – 1 000 mg/L
Colour on Hazen Scale	1 000 – 6 000 Units

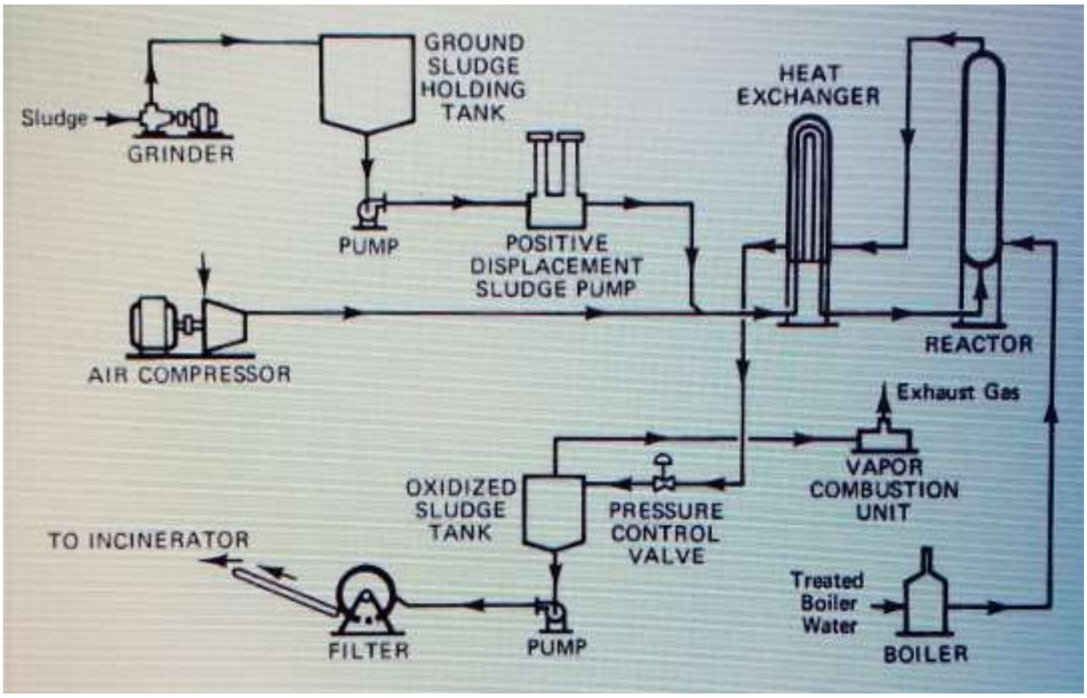


Figure 57 – TYPICAL FLOW SHEET OF THE ZIMPRO PROCESS.

Advantages and Disadvantages of the Process

Among the advantages are:

1. the sludge produced can be dewatered to a high solids content easily. On mixed sludges, up to 40% solids may be obtained;
2. no additional conditioning is required before dewatering;
3. the treated sludge is completely free of pathogenic organisms;
4. the process is not sensitive to changes in feed sludge composition;
5. being a non-biological process, it is not affected by the presence of toxic substances in the sludge;

6. no elaborate start up procedures are involved;

7. the Zimpro process can be "bottled up" over week-ends and be started up and back on stream within one hour.

Among the disadvantages are:

1. a very high capital cost, particularly if very special materials of construction are needed;

2. operation requires skilled personnel and an intensive preventative maintenance schedule;

4. the odorous gas streams must be treated before being vented;

5. the strong supernatant can cause a severe load on the treatment works;

6. maceration of the sludge being fed into the process must be very good to prevent blockages.

13.6 LOW TEMPERATURE TREATMENT PROCESSES.

13.6.1 Introduction.

A pathogen or pathogenic agent is any biological species that can cause disease in the host organism. These organisms or agents fall into four broad categories: viruses, bacteria, parasites and fungi.

The numbers and types of pathogens present in the water and the sludge differ widely and depend on many factors. The numbers of pathogens are reduced during the various treatment processes, some processes being more effective than others.

The sludge stabilization processes covered before are intended to reduce putrescibility, reduce the mass and to improve the dewaterability of the sludge. The heat treatment processes covered above will produce a sterile sludge with all pathogens killed or inactivated. Lime stabilization is not so effective and aerobic and anaerobic digestion is even less effective. As may be expected the more vigorous the stabilization process; the better the disinfection.

13.6.2 The Pasteurization Process.

It has been known for some time that heat will inactivate microorganisms as well as the cysts and ova of parasites. Different species show different sensitivities to the elevated temperatures and duration of exposure.

The critical requirement for pasteurization is that all sludge be held above a predetermined temperature for a minimum time period. Heat transfer can be accomplished by steam injection or with external or internal heat exchangers or both. Steam injection is preferred because heat transfer through the sludge layers is slow and undependable. Incomplete mixing will either increase heating time, reduce process effectiveness, or both. Overheating or extra detention are not desirable, however, because trace metal mobilization may be increased, odour problems will be exacerbated, and unneeded energy will be expended. Batch processing is preferable to avoid re-inoculations if short circuiting occurs.

The main components of a pasteurisation plant include a steam boiler, a preheater, a sludge heater, a high-temperature holding tank and storage basins for the untreated and treated sludge. Sludge for pasteurization enters the preheater where the temperature is raised from 18ºC to 38ºC through a heat exchanger than in turns cools the out flowing sludge. 30 to 40 percent of the total required heat is thus provided by recovery. Next, direct steam injection raises the temperature to 70ºC in the pasteurizer where the sludge resides for at least 30 minutes. Finally the sludge is transferred to the storage tank via the heat exchanger that heats the incoming sludge.

The pasteurization of sludge often tends to spoil the dewatering properties of the sludge. One recommended procedure is to pasteurize sludge before anaerobic digestion.

13.6.2 Composting.

Composting is considered here as a heat process because a major aim of sludge composting operations is to produce a pathogen-free compost by achieving and holding a thermophilic temperature. Available data indicate that a well-run composting process greatly reduces the numbers of primary pathogens. However, windrow or aerated pile operations have not always achieved a sufficiently uniform internal temperature to inactivate all pathogens. Adverse environmental conditions, particularly heavy rains, can significantly lower composting temperatures. An additional problem with composting is the potential regrowth of bacteria. This is particularly true with windrows where mixing moves material from the outside of the mound to the centre. However, storage of compost for several months following windrow or pile composting helps to further reduce pathogen levels.

WASTEWATER TREATMENT.

PART 14

DEWATERING AND DISPOSAL OF WASTEWATER SLUDGES.

14.1 INTRODUCTION.

Here dewatering is taken as increasing the solids content to a value in the range 10 - 30% and drying to increasing the solids content to more than 30%.

Dewatering and drying reduces the volume of sludge to be handled for ultimate disposal. The various processes are covered below.

An ideal dewatering or drying process would capture practically all the solids in the dewatered cake at minimum cost. The resultant cake would have the physical handling properties and moisture content optimal for subsequent processing. Process stability, ease of operation and compatibility with the plant environment would also be optimum.

14.2 CONDITIONING.

All wastewater sludges including those stabilized by one or more of the processes covered earlier contain substantial quantities of water bound up the gel structure in and around the sludge particles. With this hydration, the layers of water bind to the particle surface, providing a buffer, which prevents close particle approach. In addition, the solids present in wastewater are usually negatively charged and thus tend to be mutually repulsive.

The purpose of conditioning is to overcome the effects of hydration and electrostatic repulsion. It may be seen that this is similar to the coagulation process used in water treatment. See the ***Process Controller's Guide to Water Sources and Water Treatment*****,** for more information.

Conditioning, as applied to wastewater solids, is a two-step process consisting of destabilization and flocculation. In destabilization, the surface characteristics of the particles are altered so that they will adhere to one another. This change is brought about through the use of natural polymeric material secreted by the activated sludge organisms, synthetic organic polymers or inorganic metal salts. As in water treatment, flocculation is the process of providing contact opportunities, by means of mild agitation, so the destabilized particles may come together.

The amount of conditioning required for sludges is dependent on the processing conditions to which the sludge has been subjected and is usually found by trial and error.

The optimum dosage, type or types and dosage point can only be determined by trial and error. Some indication of dosage and type of conditioner can be got from laboratory tests.

The chemicals that may be used for conditioning have been adequately covered in the ***Process Controller's Guide to Water Sources and Water Treatmen***t. Understandably much larger dosages are used, and for this reason the poly-electrolyte synthetic polymers are widely used. They have the major advantages of being more shear resistant and by virtue of their low dosages 1 - 6 kg/ton dry solids, add only a small mass to total sludge to be handled.

14.3 SAND DRYING BEDS.

14.3.1 Introduction.

This is the most widely used method of sludge dewatering. They consist of a bed of sand onto which the sludge is spread. Below the layer of sand, there will be a drainage system that will remove the water that has passed from the sludge and through the sand bed. This drainage system will usually consist of one or more layers of stone, increasing in size as one goes downwards, through which the water will pass before entering a pipe for removal. The pipe system may consist of slotted pipes.

Sand drying beds may be cleaned by hand or by mechanical means and may be constructed in the open or with a roof. The drying time for the sludge is very variable depending on the type of sludge, the ambient temperature, the depth of the sludge on the bed, wind, etc. Even under similar climatic conditions, different sludges can have vastly different drying times. Drying times will usually lie in the range 3 days to 1 month. If the sludge is removed too early, extra mass in the form of water must be removed and the sludge tends to stick to the equipment being used to lift the sludge.

Removal of wet sludge can result in excessive loss of sand from the bed. This sand will have to be replaced. Unfortunately, not any sand can be used. If the effective size of the sand is less than about 0.1 mm, the slow rate of drainage through the sand retards the process. A low coefficient of uniformity is also undesirable as the small particles tend to fill up the interstices and so hinder the passage of water.

Among the advantages of sand drying beds are:

1. they have the lowest capital cost if the cost of the land is excluded;
2. they require only a small amount of skilled attention;
3. low energy consumption;
4. less sensitive to sludge variability;
5. low to no chemical consumption;
6. can produce a higher cake solids than any other process up to about 60% solids.

Among the disadvantages are:

1. needs a large area of land;
2. requires a well stabilized sludge;
3. very much weather dependent;
4. usually labour intensive;
5. the possibility of odour problems does exist;
6. need to add new sand to beds at intervals. This can be expensive if suitable sand is not available locally.

14.4 CENTRIFUGES.

14.4.1 Introduction.

There are many types of centrifuges, but only solid bowl decanter centrifuges are used for wastewater sludges.

However, with advanced design, particularly in the entrance configuration and added by very high molecular weight polyelectrolytes (M.W. of about 10 million), a high quality centrate with reasonable cake solids may be obtained. The handling capacity of the machines can vary from 1 to 25 litres per second.

There are two basic designs of centrifuge, co-current and counter-current. In the co-current design, the sludge enters at the large diameter end and is transported to the small diameter end from which it is ejected.

In the counter-current design, the sludge enters at the small diameter end and is ejected from the same end. There appears to be little to choose between the two systems. The cross-section of a typical continuous counter-current centrifuge is shown in figure 58.

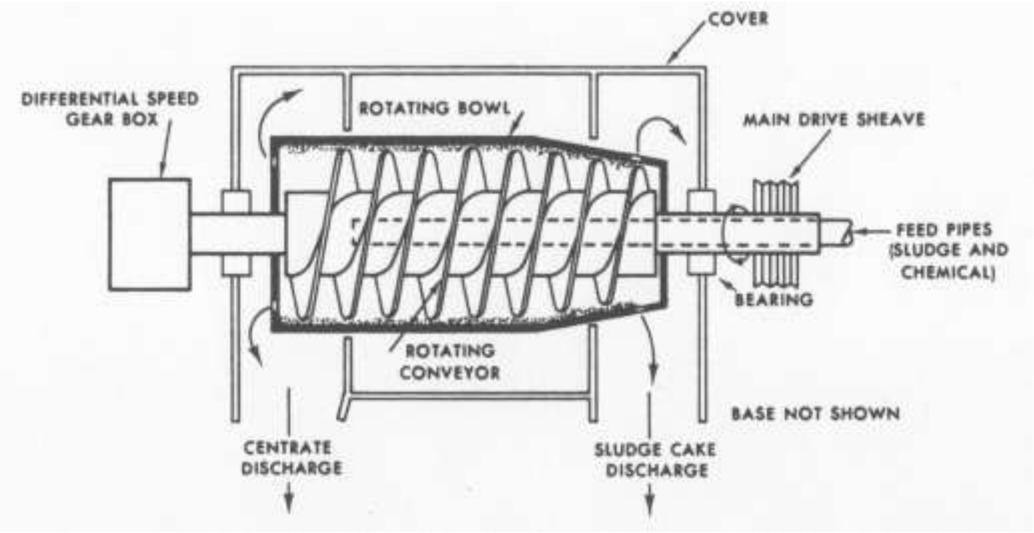


Figure 58 – SCHEMATIC OF A COUNTER-CURRENT SOLID BOWL CENTRIFUGE.

An indication of the performance that may be expected, together with the chemical change is given below. In the vast majority of cases, polyelectrolytes are used to condition the sludge in preference to metal salts as shear resistance is important.

TABLE 18 – TYPICAL VALUES WITH A SOLID BOWL CENTRIFUGE.

SLUDGE TYPE	FEED SOLIDS % Dry Solids	CAKE SOLIDS % Dry Solids	POLY. DOSE Kg/ ton Dry Solids	SOLIDS RECOVERY %
Raw Primary	5 - 8	25 - 35	1 - 2	70 - 90
Digested Primary	5 - 8	28 - 35	2 - 5	95+
Waste Activated	0.5 – 2.5	8 - 12	2 - 6	85 - 95
High Lime	10 - 12	30 - 50	nil	90 - 95
Anaerobically Digested Pri + WAS	4 - 7	15 - 20	2 - 5	70 - 95

Among the advantages of using a solid bowl decanter centrifuge are:

1. a clean compact process with quick start up and shut down procedure;
2. does not require continuous operator attention;
3. may be installed in the open;
4. little or no odour problems;
5. no wash water required, except during shut down.

Among the disadvantages are:

1. requires skilled maintenance;
2. for waste activated sludge at least, the cake solids is not very high;
3. feed stream should be macerated.

14.5 VACUUM FILTERS.

14.5.1 Introduction.

In vacuum filtration, atmospheric pressure due to a vacuum applied downstream of the media, is the driving force on the liquid phase that moves it through the porous media. Figure 59 shows the cutaway view of a typical rotary vacuum filter. The covering of the filter may be natural or synthetic fibre cloth or woven stainless steel cloth or a blend of the two.

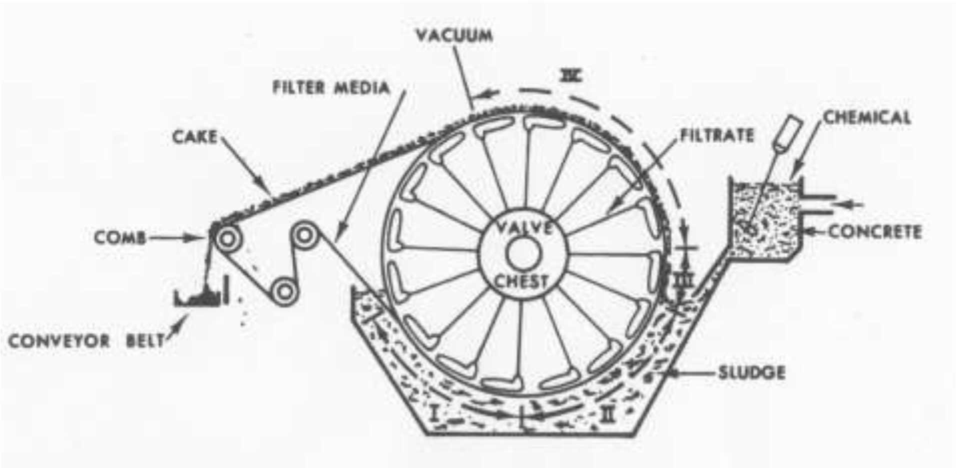


Figure 59 – CROSS SECTION OF A TYPICAL VACUUM FILTER.

An indication of the performance that may be expected is given below in table 19:

TABLE 19 – TYPICAL VALUES WITH A VACUUM FILTER.

SLUDGE TYPE	FEED SOLIDS % Dry Solids	CAKE SOLIDS % Dry Solids	POLY. DOSE Kg/ ton Dry Solids	SOLIDS RECOVERY %
Raw Primary	4 - 8	20 - 40	2 - 3	75 - 95
Digested Primary	4 - 8	30 - 50	2 - 3	80 - 95
Waste Activated	0.5 – 2.5	5 - 10	2 - 5	85 - 95
Anaerobically Digested Pri + WAS	4 - 7	20 - 35	2 - 4	80 - 95

Among the advantages of the process are:

1. does not require skilled personnel;
2. has low maintenance requirements;
3. the filtrate has a low suspended solids content;
4. can be rapidly started up or shut down;

1. has the greatest power demand of all dewatering processes except heat drying;
2. requires continuous operator attention;
3. the vacuum pumps are usually very noisy;
4. blinding of the filter cloth can occur.

14.6 BELT PRESSES.

14.6.1 Introduction.

The belt press has three operational stages: chemical conditioning of the feed slurry, gravity thickening to a non-fluid consistency, and compaction of the thickened sludge.

Good chemical conditioning is the key to successful and consistent performance of the belt filter press, as it is for other dewatering processes.

After conditioning, the readily drainable water is separated from the slurry by discharge of the conditioned material onto the moving belt in the gravity drainage section. Typically, one or two minutes are required for drainage. Following drainage, the sludge will have a dry solids content of about 5 - 7%.

The formulation of an even surface cake at this point is essential to the successful operation of subsequent stages of the dewatering cycle. The even surface prevents uneven belt tension and distortion while the relative rigidity of the mass of sludge allows further manipulation and gives maximum speed through the machine.

The third stage of the belt press begins as soon as the sludge is subjected to an increase in pressure, due to the compression of the sludge between the carrying belt and cover belt.

Some designs have separate belts for the free-drainage and dewatering stages whereas others use the same belt for both stages. Separate belts means that the belt in the free drainage stage can have a coarser weave to allow for faster drainage.

During pressure application, the sludge cake, squeezed between the two belts, is subjected to flexing in opposite directions as it passes over the various rollers. This action causes increased water release and allows greater compaction of the sludge. Generally the more rollers, the better the degree of dewatering. Figure 60 shows the various parts of a typical belt press unit.

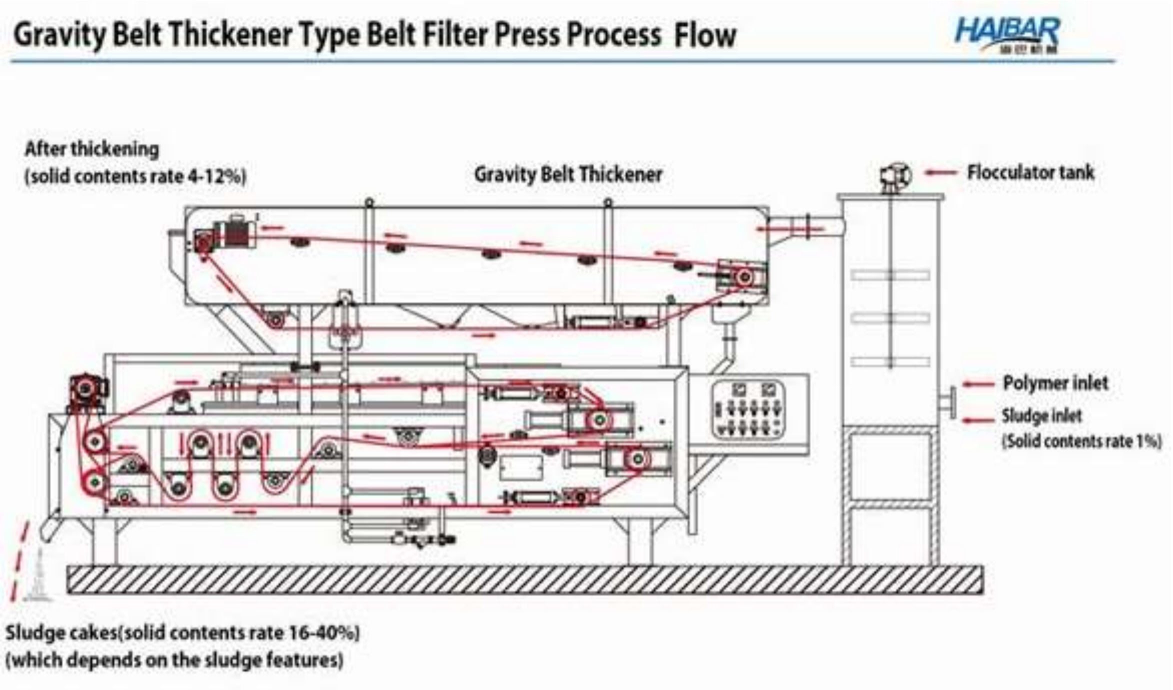


Figure 60 – LAYOUT OF A TYPICAL BELT PRESS.

TABLE 20 – TYPICAL VALUES WITH A BELT PRESS.

SLUDGE TYPE	FEED SOLIDS % Dry Solids	CAKE SOLIDS % Dry Solids	POLY. DOSE Kg/ton Dry Solids	SOLIDS RECOVERY %
Raw Primary	4 - 8	25 - 40	1 - 4	95
Digested Primary	4 - 8	25 - 40	1 - 4	95
Waste Activated	0.5 – 2.5	13 - 17	2 - 6	80 - 90
Anaerobically Digested Prim + WAS	4 - 7	20 - 30	2 - 4	95

Among the advantages of the process are:

1. high cake solids;
2. low power requirements;
3. continuous flow process.

Among the disadvantages of the process are:

1. very sensitive to incoming feed characteristics
2. machines hydraulically limited in throughput;
3. short media life when compared with other processes using cloth media;
4. wash water needed continuously. Can result in significant return of solids adversely affecting overall solids recovery;
5. blinding of the media can occur if insufficient washing of the filter belt.

14.7 FILTER PRESSES.

14.7.1 Introduction.

All the various dewatering processing detailed above are operated on a continuous basis. The filter press (usually called the Plate and Frame Press) is operated on a batch-wise basis. The operation can be automated.

The unit consists of a number of parallel plates covered on both sides, with filter cloths and shaped to permit drainage. The side view may be seen in figure 61. The sludge is introduced under pressure into the spaces separating successive plates and the water is forced through the filter cloths, leaving behind the solids. After a suitable period the press is opened and the solids removed.

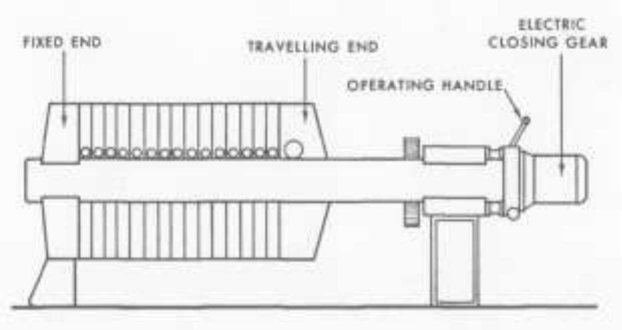


Figure 61 – SIDE VIEW OF A FILTER PRESS (PLATE and FRAME).

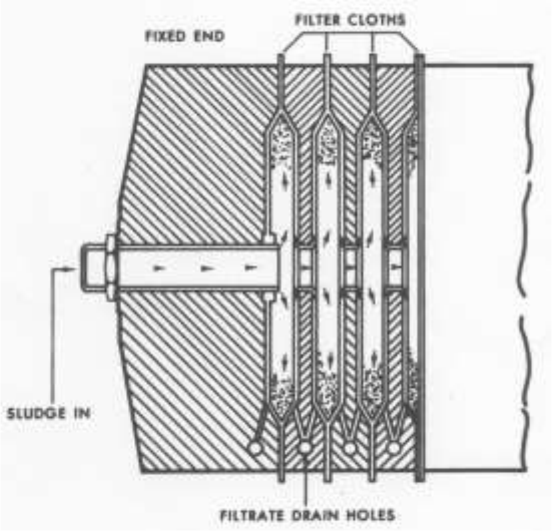


Figure 62 – DETAILS OF A FILTER PRESS (PLATE and FRAME).

An indication of the performance of the process is given below in table 21

TABLE 21 – TYPICAL VALUES WITH A FILTER PRESS (PLATE and FRAME).

SLUDGE TYPE	FEED SOLIDS % Dry Solids	CAKE SOLIDS % Dry Solids	CHEMICAL. DOSE (see note)	SOLIDS RECOVERY %
Raw Primary	4 - 8	45	1 - 4	95+
Digested Primary	4 - 8	50	1 - 4	95+
Waste Activated	0.5 – 2.5	45	2 - 6	95+
Anaerobically Digested Pri + WAS	4 - 7	45	2 - 4	95+

NOTE: The chemical dosage in the above example was 5% Ferric Chloride and 10% Lime by mass

Among the advantages of the Process.

1. the very high cake solids achieved. It must be noted however that this is in part due to the very high chemical doses, up to 150 kg/ton dry solids of sludge. The actual cake solids in terms of feed sludge, is much lower;
2. the dewatered sludge may be autothermic when burnt in a suitable incinerator and it may be possible to re-use the calcium oxide recovered from the ash.

Among the disadvantages of the process are:

1. it is a batch operation. However, the system can be automated;
2. can have a high labour cost;
3. has high capital cost;
4. has a large area requirement;
5. high chemical dosage required.

14.8. HEAT DRYING.

Heat drying is the process of evaporating water from sludge by thermal means. With the possible exception of well dried sand bed dewatered sludge, the cake produced from the other processes covered above is not really dry. Only in relatively few cases will the solids content be above 40%, this means that 60% or nearly two thirds of the sludge is water.

In order to reduce the moisture content to the 20 - 30%, a thermal method is required. The costs of thermal drying are high and can only be justified when the dried product can be sold at cost or at a slight profit. In terms of the nitrogen and phosphorus contents of the sludge, thermally dried sludge cannot compete with inorganic fertilizers.

Unless there is a source of waste heat such as from an incinerator or pyrolysis plant, the thermal drying process appears to be no longer economical.

14.9 SLUDGE MASS REDUCTION.

14.9.1 Introduction.

In the stabilization process covered earlier, the maximum reduction in total solids has been of the order of 30 - 40%. (The volatile solids reduction has been of the order of 40 - 60%). There are processes available that can reduce the mass of sludge by a much larger amount. Two of these processes are incineration and starved air combustion or S.A.C. (also known as pyrolysis).

14.9.2 Incineration.

This is the combustion of sludge to gases and a residual ash. The quantity of heat required to vaporize the water from unit mass of sludge is substantial. Only in certain cases, will sufficient heat be available from the combustion of the sludge to match the heat requirement. When no supplemental fuel supply is required for the main combustion process, the feed sludge is said to be auto-thermic. Usually additional fuel is required to fire the afterburner which is required to ensure that the exhaust gases are free of unburnt material.

Incineration is a two-step process involving drying and combustion. In addition to fuel and air, time, temperature and turbulence are necessary for a complete reaction. The drying step must not be confused with the preliminary dewatering. This dewatering is usually by mechanical means and precedes the incineration process in all systems.

The drying and combustion process consists of the following phases:

1. raising of the temperature of the feed sludge to 100ºC;
2. evaporation of the water from the sludge;
3. increasing the temperature of the steam and the air;
4. increasing the temperature of the dried sludge volatiles to the ignition point.

The practical operation of an incinerator requires that air in excess of the theoretical requirements be supplied for complete combustion of the sludge and the supplementary fuel. The introduction of this excess air has the effect of reducing the burning temperature and increasing the heat losses from the incinerator.

14.9.3 The Multiple Hearth Furnace

This is the most widely used type of sludge incinerator. It is simple, durable and has the flexibility of burning a wide variety of materials even with fluctuations in the feed rate. The furnace consists of a circular steel shell surrounding a number of solid refractory hearths and a central rotating shaft to which rabble arms are attached. The arms provide a mixing action as well as a rotary and downward movement of the sludge. Air is blown into the system to cool the rabble arms and at the same time being pre-heated before being discharged into the combustion area of the furnace.

A large percentage of the operating costs is attributable to fuel. This type of unit takes about 24 hours to heat up from ambient to operational temperature and should be cooled over about the same period. They should therefore be run continuously. A cross section is shown in figure 63.

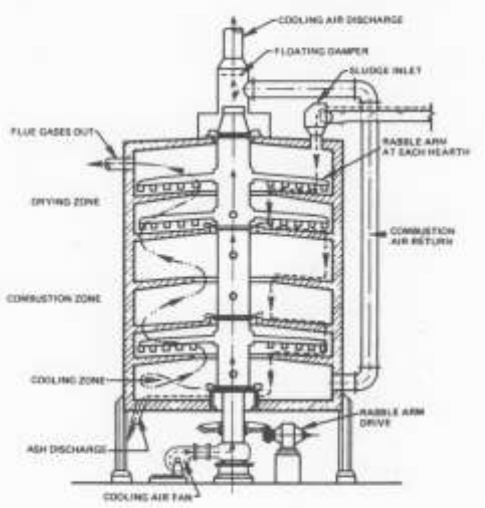


Figure 63 – CROSS-SECTION OF A TYPICAL MULTIPLE HEARTH INCINERATOR.

14.9.4 The Fluidized Bed Incinerator.

This is a vertical cylindrical vessel with a grid in the lower section to support the sand. Dewatered sludge is injected above the grid. Air is blown upwards through the grid and fluidizes the mixture of hot sand and sludge. Supplemental fuel may be injected above the grid.

Because a very large quantity of heat is stored in the sand, the incinerator may be operated on a 5-day per week basis. A fairly short heating up period will be needed on Monday mornings. Once every one to two months it will be necessary to add some sand to make up for losses. A cross section is seen in figure 64.

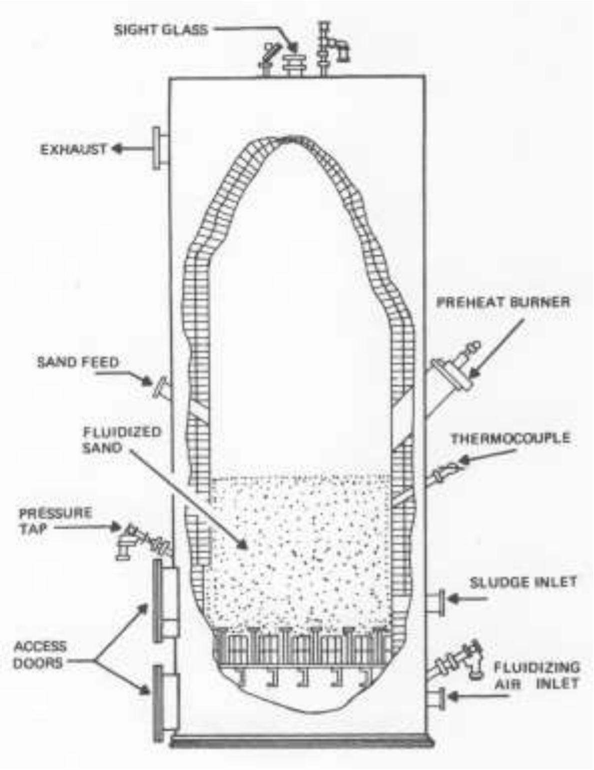


Figure 64 – CROSS-SECTION OF A TYPICAL FLUIDISED BED INCINERATOR.

14.9.5 Starved Air Combustion or S.A.C. (Pyrolysis).

This is a modification of the incineration process where less than the theoretical amount of air is admitted to the incinerator allowing some unburnt material to be recovered for use as a fuel elsewhere. It will require a specially designed afterburner to ensure a clean exhaust gas.

This process is very difficult to manage to ensure that the correct amount of air is admitted to the unit. Too much air and the contents can catch fire and too little air results in the process being incomplete.

14.10 ULTIMATE SLUDGE DISPOSAL.

14.10.1 Introduction.

The ultimate disposal of the solid fractions derived from the treatment of wastewater can be one of the most expensive operations in the whole treatment process.

It is most important that the solid fractions, i.e. sludge, grit and screenings as well as ash from incineration, be disposed of in such a manner as to avoid secondary pollution. There is only one final disposal area and that is the land. The operations that can be performed on the land are known as land-spreading or landfilling. Disposal at sea is not permitted.

14.10.2 Land-spreading.

This, as its name suggests, involves the spreading of sludge on the surface of the land. In this respect, the surface of the land is taken to mean the top 300mm or so.

The sludge may be spread as a liquid, semi-solid, solid or as the incinerated residue. In other words sludge may be spread in any form containing between 0.5% solids and 95% solids. The sludge may be sprayed onto the ground, injected into the ground or scattered on the ground. In most cases it is usual to plough or disc hammer to bury the sludge.

Sludge is a useful soil conditioner that helps to break up heavy soils and improves the moisture retaining ability of sand soils. Although the nitrogen, phosphorus and potassium contents of all municipal sludges are low when compared with inorganic fertilizers, these nutrients are not leached out as quickly as those from the inorganic fertilizers. Most sludges contain large quantities of humic material that is advantageous. Sludges derived from domestic wastewater treatment works contain trace elements and minor nutrients in relative concentrations that are of the same order as required by most crops. This means that provided reasonable spreading rates are used one should not experience trace element problems with domestic sludges.

However, when industrial effluents are present in the wastewater, significant concentrations of metals and other undesirable constituents may be present in the sludge and it is most important to make sure that the spreading rates are such that undesirable concentrations are not encountered. Just what is an undesirable concentration is not easy to define. There are many factors to be considered, such as the type of soil, rainfall or irrigation quantities, the type of crop to be grown and synergistic or antagonistic effects of other materials present in the sludge or in the ground.

Sludge is often composted with a variety of other materials before being spread on the land. This can have several advantages including reducing the metal concentration of the sludge and producing a material that is more easily incorporated into the ground.

2020 UPDATE. A series of guidelines dealing with the ultimate disposal and use of wastewater sludge have been published by the Water Research Commission.

14.10.3 Landfill.

Sludge may be disposed of by landfill, alone or in combination with other solid wastes. Landfills have been fully covered in the ***Process Controller's Guide to Pollution Control***.

Screenings and grit removed from wastewater are nearly always disposed of by landfill. Where incinerators are in use, they may be incinerated first.

When sludges from works treating industrial effluents were spread on the ground, one needs to make sure that undesirable concentrations of certain materials are not being built up in the ground. This could limit the spreading rate to such a low value that little benefit could be obtained from the beneficial components of the sludge. This could also mean that very large areas of land were needed. In landfill, however, this problem does not arise provided the landfill site is well prepared and the accepted tipping procedures are strictly adhered to, and the site is licenced to accept wastewater sludge.

14.10.4 Operational Aspects of Land Disposal.

The disposal of sludge onto land may not be possible at certain times of the year due to climatic effects such as heavy rain, snow, frost, etc. and in the case of land spreading due to the use of the land for growing crops.

As the production of sludge at a treatment works is essentially a continuous process, some storage capacity will have to be incorporated somewhere in the treatment / disposal route. Among the storage facilities are:

1. in the sludge digesters;

2. in deep lagoons;

3. in holding tanks;

4. in heaps.

WASTEWATER TREATMENT.

PART 15

THE INTERPRETATION OF ANALYTICAL RESULTS.

15.1 INTRODUCTION.

In order to monitor the effectiveness of the treatment process, it is necessary to take samples of the feed, in-process and effluents and to analyse these samples for certain parameters. In this section, the interpretation of the results of analysis is given.

The test methods are not given here as they are prescribed in the various standard methods that have been published elsewhere.

The tests performed in the laboratory may be divided into 4 sections:

1. physical tests that measure and record the properties detectable by the senses;
2. chemical tests that determine the amounts of mineral and organic material present in the water at the various stages of treatment. These are used to determine the loading to be treated by the works and the efficiency of treatment at the various stages;
3. bacteriological examinations indicate the survival of pathogenic bacteria through the treatment process and give an indication of the impact of the discharge of the treated effluent into the receiving waters;
4. microscopic examination provides information concerning the micro-organisms present in biological filters and the activated sludge system.

The publication "Standard Methods for the Examination of Water and Wastewater" published jointly by the A.P.H.A., the A.W.W.A and the W.P.C.F. is generally accepted as the reference book. Certain analytical methods have been promulgated in the Government Gazette in this country. It is important that the standard procedures be strictly adhered to so that:

1. the effects of interferences are known;
2. results obtained in one laboratory can be directly compared with those obtained in another laboratory.

15.2 PHYSICAL TESTS FOR WASTEWATER.

15.2.1 Colour.

Normal domestic wastewater is usually a brownish-grey colour when fresh or stale; when it is septic it becomes black. Night soil is usually an orange-brown colour. The presence of certain industrial wastes can colour the wastewater almost any colour. Dyestuffs, for example, can make the wastewater pink, green, blue, etc. Blood from abattoirs can stain the wastewater red.

15.2.2 Odour.

Fresh domestic wastewater has very little smell, as it becomes septic the smell increases as the result of formation of sulphides and mercaptans. The presence of certain industrial effluents can affect the odour. Milk waste discharges will give off a sour, lactic acid smell, while solvents will have their own characteristic odour.

15.2.3 Appearance.

The appearance of the wastewater will change depending whether it has been pumped or not, as pumping breaks up the solids. High detergent loads will cause foaming while oils and other lighter-than water materials will float on the water. Heavy infiltration of groundwater or rainwater will dilute the wastewater and this may be seen.

15.2.4 General.

It is important for the Process Controllers on shift to regularly look at the raw, in-process and treated streams. It is usually worthwhile taking a sample for analysis when a sudden change occurs in the appearance, etc., of the water. The analysis may help to locate the cause of the change. Included in this is belching in primary sedimentation tanks resulting in suspended solids being carried over the weir to the biological filters or the aeration tank. This will increase the load on these units and this may result in nitrification stopping. The carry-over of solids from a secondary sedimentation tank may result in a build-up of solids in the maturation ponds or shorter filter runs if the sedimentation tank effluent is sand filtered.

15.3 CHEMICAL TESTS FOR WASTEWATER.

The test methods are not given here as they will be found in the various standard method publications. The significance and usefulness of the tests will be discussed.

15.3.1 Tests to determine the amount of oxidizable material in the influent, in-process and treated wastewater.

These results given an indication of the strength of the wastewater at various stages in the treatment process. They also tell one, how effective the treatment process is. In the case of the final effluent, it tells one whether the final effluent complies with the required standard.

15.3.1.1 The Chemical Oxygen Demand - COD

This was covered in section 1.5.4

15.3.1.2 The Biochemical Oxygen Demand or BOD.

This has been covered in section 1.5.5.

15.3.1.3 Permanganate Value - 4h – (PV4).

This was covered in section 1.5.7

15.4 TESTS THAT INDICATE THE PRESENCE OF NUTRIENTS IN THE WASTEWATER.

15.4.1 The Various forms of Nitrogen.

15.4.1.1 Free and Saline Ammonia (Ammonia Nitrogen).

These are the most simple reduced forms of nitrogen in water and wastewater. The more complex compounds of nitrogen such as proteins and amino acids are broken down into ammonia, by biological action. The presence of ammonia in natural waters usually means that pollution due to domestic or animal wastes has taken place.

The ammonia content of wastewater has a significant bearing on the ultimate oxygen demand of the water. It will be remembered that the COD test does not indicate the oxygen demand due to nitrogenous material in the wastewater. It can be shown that 1 mg/L ammonia as nitrogen has an oxygen demand of 4.6 mg/l oxygen.

Low contents of ammonia are usually determined by the colourimetric method and above about 1 mg/L by the titrimetric method. The ammonia specific ion electrode may be used at higher values.

15.4.1.2 Kjeldahl Nitrogen.

This test determines the more complex forms of nitrogen such as proteins and amino acids. When the sample is boiled with sulphuric acid, all the carbon in the water is oxidised to carbon dioxide by the sulphuric acid, the latter being reduced to SO_2. All reduced nitrogen containing material is transformed into ammonium hydrogen sulphate NH_4HSO_4. After making alkaline, the ammonia is distilled off and determined as above. Any ammonia originally present in the sample shows up in the analysis. The total result obtained is known as the Total Kjeldahl Nitrogen or TKN. When the Ammonia Nitrogen content is subtracted, the difference is known as the Kjeldahl Nitrogen (KN) or Organic Nitrogen.

Nitrite and Nitrate and dissolved Nitrogen gas do not show up in this test as they are boiled off after the addition of the sulphuric acid as nitrous acid, nitric acid and nitrogen gas respectively.

The sum of all forms of nitrogen is known as the Total Nitrogen - TN.

15.4.1.3 Nitrite and Nitrate.

When ammonia is oxidised, the first step in the reaction is the formation of nitrite, this is the first stage of nitrification.

$$2NH_4^+ + 3O_2 \rightarrow 2NO_2^- + 2H_2O + 4H^+ + 2e$$

The second step of the nitrification reaction is the oxidation of nitrite to nitrate.

$$NO_2^- + O_2 \rightarrow 2NO_3^-$$

The first step is performed by the Nitrosomonas bacteria, is usually the slower reaction so that when the nitrite is formed it is rapidly oxidised to nitrate by the Nitrobacter bacteria. It is unusual to find nitrite contents greater than about 1 mg/L as N; usually the content is less than 0.5 mg/l as N. The nitrite colourimetric method using diazotization is a very sensitive reaction and very low concentrations may be detected.

When the nitrite plus nitrate is reduced with Devarda’s alloy, the ammonia formed may be distilled over and detected as indicated above. It is usual to determine the ammonia first, add distilled water and Devarda’s alloy and then to distil off the ammonia derived from the nitrite and the nitrate. Since this method determines nitrite and nitrate while nitrite is determined separately, the nitrate content is determined by difference. There are methods available to determine nitrate only.

15.4.2 The various forms of Phosphorus.

Phosphorus is usually what is known as “growth limiting nutrient” in receiving waters. In other words, the amount of phosphorus in the receiving water has a major influence in the growth of aquatic weeds, algae etc. This is the reason why it is important to reduce the amount of phosphorus in the treated effluent. This aspect was covered in the earlier ***Process Controllers Guides***. This is a good example of where wastewater treatment has an effect on water sources.

15.4.2.1 Total Phosphorus.

This includes all forms of phosphorus such as: ortho-phosphate, condensed phosphates and organically combined phosphorus. As the analytical methods used determine only ortho-phosphate, the other forms of phosphorus must first be converted into ortho-phosphate. To do this, the sample is digested in a strongly acid medium. Perchloric Acid may be used, but this is very dangerous. A mixture of sulphuric and nitric acids is usually suitable.

15.4.2.2 Ortho-Phosphate.

Here the colour development is performed on the unhydrolysed sample, so that only ortho-phosphate is determined. It is particularly important in all phosphorus determinations that detergents be avoided if at all possible. If necessary, glassware can be cleaned with phosphate-free detergent. It is preferable to keep a separate set of glassware for phosphorus determinations and to use hot 10% Hydrochloric or sulphuric acid for cleaning.

15.5 TESTS THAT INDICATE THE PRESENCE OF MINERALS IN THE WATER.

15.5.1 Total Dissolved Solids (TDS).

This may be determined by filtering a sample, taking a known volume of the filtrate, evaporating the water and determining the mass of the residue. An alternative method is to determine the total solids and to subtract the suspended solids. (See below)

The size of the pores in the filter paper will have an effect on the result obtained as a coarser paper will allow more undissolved material to pass through into the filtrate. It appears as though the Whitman's glass fibre paper GF/A is becoming the standard filter paper. The temperature, at which the sample is dried, has an effect on the result obtained. Two standard temperatures are used 180ºC and 103ºC. The higher is usually used in water analysis and the lower for wastewater analysis, because of the organic content of the latter.

15.5.2 Total Inorganic Dissolved Solids (TIDS).

When the residue from the above determination is heated to 550ºC, the organic materials present will decompose and volatilize and leave the inorganic solids behind. The material left after ignition is known as the inorganic or non-volatile residue, the mass of material lost during ignition is known as the organic or volatile matter.

15.5.3 Electrical Conductivity.

This is a measure of the water's ability to convey an electric current. This property is related to the total concentration of ionized dissolved substances in the water and to the temperature of the water. It is only the ionized substances that will conduct electricity in the water. The conductivity is the reciprocal of the electrical resistance. The standard unit of electrical conductivity is the siemens/metre. The pre - S.I. unit was the reciprocal ohm-cm or mho-cm. When the SI system was introduced, the electrical conductivity was reported as µS/cm – because it was numerically the same as mho-cm. As this is not consistent with the metre being the standard distance unit is the SI system, **this method of reporting electrical conductivity should no longer be used.** When looking at results – it is important to note the form in which the result is given.

10µS/cm = 1mS/m

If a direct current voltage is used to determine the electrical conductivity, then cations will move to the cathode and anions will move to the anode and polarization will occur at the electrodes due to the liberation of gases. Today alternating current is used and this overcomes the effects due to polarization.

The electrical conductivity is often used as a quick check on the dissolved solids content of a sample. The conversion factor from electrical conductivity to total dissolved solids varies depending on the soluble components of the water. For domestic wastewater at 25ºC, the factor is about 6.5. That is 1 mS/m is approximately equal to 650 mg/L of inorganic dissolved solids.

15.5.4 Chloride

Chloride is one of the major anions in water and wastewater. The salty taste produced by chloride concentrations is variable and dependent on the chemical composition of the water. Some waters containing 250 mg/L Chloride may exhibit a detectable salty taste in the presence of sodium ions.

On the other hand, the typical salty taste may be absent in waters containing as much as 1 000 mg/L chloride when there is a predominance of calcium and magnesium ions. Infiltration of saline groundwater or sea water into sewers will increase the chloride content of wastewater. Certain industrial effluents may have high chloride contents. A high chloride content in water has a deleterious effect on agricultural plants.

15.5.5 Sodium.

Sodium is one of the major cations in wastewater. Usually most of the sodium found in water and wastewater is associated with chloride. The ratio of sodium to total cations is important in agriculture and human pathology. Soil permeability has been found to be detrimentally affected by a high sodium ratio in the water.

15.5.6 Sulphate

This is a widely distributed anion. It has a lesser deleterious effect than has chloride on the agricultural use or re-use of water.

When wastewater becomes septic, sulphate-reducing bacteria metabolise sulphate as a source of oxygen and liberate sulphide. The latter is toxic to man, has an unpleasant smell, and can be oxidised by certain bacteria to form sulphuric acid. This can cause the corrosion of concrete.

15.5.7 Calcium and Magnesium.

These are hardness causing substances and are covered in the ***Process Controller's Guide to Water Sources and Water Treatment.***

15.6 MISCELLANEOUS OTHER PARAMETERS.

15.6.1 Settleable Solids.

This is a measure of the readily settleable material in raw wastewater or in-process wastewater. For raw wastewater, settled wastewater, bio-filter effluent and secondary sedimentation tank effluents an Imhoff cone is used and 60 minutes is allowed for settlement. For use in the activated sludge process, a straight sided measuring cylinder is used and 30 minutes is allowed for settlement. This result is used in calculating the Sludge Volume Index – see section 7.11.3.

15.6.2 Suspended Solids.

This is a measure of the non-dissolved material present in a water.

There are two methods for determining the suspended solids. One method is for low values and the other for the higher values.

1. for low values, one determines the suspended solids directly. This is done by filtering a known volume through a filter paper and determining the increase in the mass of the filter paper due to the suspended solids. One then calculates the mass per litre as mg/L;

2. for higher values, one determines the suspended solids by difference. The total solids of a 100mL sample is determined. The total dissolved solids of another 100mL sample is determined. The suspended solids is then **total solids** minus the **total dissolved solids**. This is then reported as mg/L.

The former test will be used for secondary sedimentation tank effluent, while the latter will be used for raw and settled wastewater and for determining the Mixed Liquor Suspended solids (MLSS).

The porosity of the filter paper used to hold the suspended material will have an effect on the result obtained. For comparative purposes the Whatman's GF/A glass fibre paper should be used.

This is a very important test in the activated sludge process as it is used to determine the amount of sludge in the system. The so-called Mixed Liquor Suspended Solids (MLSS). This has been covered in detail in section 7.11.3.

15.6.3 Sludge Volume Index SVI.

This has also been covered in section 7.11.3.

15.6.4 Dissolved Oxygen.

In those wastewater treatment processes that are aerobic in nature, it is essential to keep the dissolved oxygen content above the limiting value so that the treatment process may proceed unhindered. In denitrification practice, in the anoxic zones it is important that the dissolved oxygen content be kept zero. The importance of dissolved oxygen in the environment was covered in detail in the ***Process Controller's Guide to Pollution Control.***

With reliable D.O. meters available today, the Winkler titration method is becoming less widely used in wastewater treatment, particularly as it is not very satisfactory in the presence of solids that can be oxidized by the iodine. The most satisfactory method to calibrate a D.O. meter is to thoroughly aerate tap water in which the probe is placed during calibration, the temperature and pressure are noted and the saturated D.O. determined in a table, the meter is then set to that value.

15.6.5 pH Value.

This is a measure of the hydrogen ion concentration. Micro-organisms are sensitive to sudden and wide changes in pH value. Generally pH values outside the range 4.5 to 9.5 are due to man-made pollution.

15.6.6 Alkalinity.

This is a measure of the alkaline substances in solution in the water. These materials have a buffering action that resist the change in pH value when acids or alkalis are added. In water, alkalinity is due to the bicarbonate and carbonate ions, while in wastewater in addition to these two, ammonia and certain organic acids and their salts contribute to the alkalinity. The significance of alkalinity in wastewater treatment has been covered earlier in the various sections.

15.6.7 Sulphide.

Sulphide as hydrogen sulphide is generated by sulphate-reducing bacteria in septic wastewater. Besides causing corrosion when oxidised to sulphuric acid, sulphides appear to enhance the growth of certain filamentous organisms in activated sludge and these can contribute to the bulking of the sludge.

15.6.8 Heavy Metals.
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Heavy metals in excessive concentrations are toxic to the microorganisms of purification.

15.6.9 Volatile Acids.

These have been covered in Anaerobic Digestion in sections 15.2 onwards.

15.6.10 Specific Oxygen Uptake Rate (SOUR).

This test measures the activity of the organisms in the mixed liquor in the reactor of an activated sludge works. It determines the rate at which the organisms utilise dissolved oxygen. In short, the test involves aerating a sample of mixed liquor to near dissolved oxygen saturation point and then measuring the rate at which the dissolved oxygen decreases as a result of the organisms utilising the dissolved oxygen. The results are plotted on a graph and the slope of the straight line portion determined in mg/L per min.

As the mixed liquor suspended solids has affects the result, the best is to determine the mixed liquor suspended solids (MLSS) at the same time.

The results are reported as **mg/L Oxygen per 1 000 mg/L per min.** This is the Specific Oxygen Uptake Rate.

This makes it possible to compare the activity of the sludge at one treatment works with that of another treatment works.

A typical plot of Dissolved Oxygen versus Time is shown in figure 65. The oxygen uptake rate is calculated using the values at "A" and "B" as this is the straight line portion of the plot.

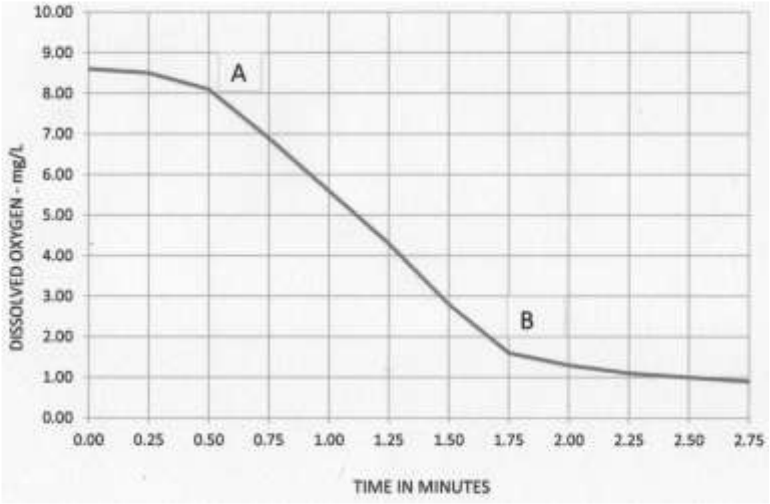

Figure 65 – TYPICAL PLOT FOR DETERMINING THE OXYGEN UPTAKE RATE.

If the treatment works is adversely affected by some material in the influent and the activity of the organisms DECREASES then the dissolved oxygen content in the reactor will INCREASE as the organisms can no longer utilise the dissolved oxygen as quickly as before. This is where some Process Controllers make the mistake of decreasing the rate of aeration, whereas they should be increasing it.

15.7 BACTERIOLOGICAL TESTS.

As has been noted before, raw wastewater contains very large numbers or organisms, some pathogenic and some non-pathogenic. During the various stages of treatment of the liquid and solid fractions, a proportion of the organisms is killed off or inactivated. The degree of die-off or inactivation depends on many factors.

In order to determine the residual content of organisms various bacteriological tests are performed. Amongst the more common are the tests for the following:

1. total coliforms;

2. Escherichia coli;

3, staphylococcus group;

4. salmonella group

The Escherichia coli are the most commonly determined. Although most E. Coli are not pathogenic (disease causing); they general survive longer than most of the pathogenic organisms during the treatment processes. This means that one can be fairly sure that when the E, Coli count is low or nearly zero, the bulk of the pathogenic organism are present in smaller numbers.

There are two basic methods of determining the concentration of specific organisms or group of organisms:

1. the membrane filtration method, where a sample of liquid is filtered through a membrane which is then placed on a nutrient pad in a petri-dish and incubated at a prescribed temperature for a prescribed period. Many samples will have to be pre-filtered through a sterile pre-filter. The content of organisms is done by counting the number of colonies present.

2. the most probable number multiple tube method where usually 3 sets of test tubes containing a specific growth medium are inoculated with small volumes of the sample to be tested. At the end of the prescribed incubation period, a certain number of the tubes will be found to be positive with respect to the presence of the wanted organism. By referring to the number of tubes of each dilution that shows a positive response, the most probable number of organisms present in the original sample may be looked up in a set of specially prepared statistical tables.

As one is not attempting to produce a sterile effluent, one is usually not concerned with non-pathogenic organisms.

15.8 MICROBIOLOGICAL TESTS.

15.8.1 Discussion.

In this context, microbiological is taken to be referring to all forms of small organisms other than the bacteria referred to in section 15.7 above. This may not be strictly correct, but it serves our purpose here. Microbiological examinations can be used as an aid to monitor an activated sludge works and as an aid in determining the degree of pollution in a body of water, e.g. into which effluent has been discharged. In most cases one examination will not indicate anything except when gross pollution is present; it is the change in the relative numbers and types of organisms that provides the useful information.

15.8.2 Microbiological Examination of Activated Sludge.

As seen earlier, activated sludge is a complex mixture of many organisms. As many organisms can compete for the same substrate, it is not a simple matter to decide which are "good" organisms and which are "bad". As the object of an activated sludge works is usually to produce a well oxidised clear fully nitrified effluent, and often as fully denitrified as possible, it does not really matter which organisms do the work. However, certain organism such as the filamentous organisms, are bulky and enmesh large amounts of water within their mass and this makes them settle very slowly. This can result in a poorly settling sludge and in order to prevent loss of sludge over the secondary sedimentation tank, it is necessary to reduce the mass of sludge in the system and this reduces the sludge age. When the works is lightly loaded, a badly bulking sludge can still produce a clear well nitrified effluent. In general the greater the numbers of higher forms of life, the higher the quality of the sludge.

15.8.3 Microbiological Examination of a Body of Water.

The effect of pollution on organisms was covered in section 2.5 in ***the Process Controller's Guide to Pollution Control***. Microbiological examinations can assist in determining the extent of each zone of degradation and recovery.

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