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Final Report to the Water Research Commission on laboratory studies on the removal of phosphate and nitrogen compounds from biological filter effluents

by

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1. SUMMARY

The research carried out under the contract dealt with laboratory experiments on chemical phosphate removal from biological filter effluents and with desk and pilot studies on the removal of nitrogenous compounds by biological means.

It is generally accepted that phosphates can only be removed chemically from humus tank effluents as biological filter systems do not provide for the necessary conditions required for biological phosphate removal. Ferric chloride was used as a precipitant in beaker experiments on the mechanism of chemical phosphate removal from wastewater. It was shown that the degree of phosphate removal depended on the speed of mixing and that the precipitation of orthophosphate was not stoichiometric. Since mostly orthophosphate is precipitated by ferric chloride at the pH common in wastewater one has to be aware that especially domestic sewage contains considerable amounts of condensed phosphates, derived mainly from synthetic laundry detergents. These condensed phosphates are not removed by $FeCl_3$, but are hydrolyzed by biological wastewater treatment, thus yielding an increase in the orthophosphate concentration in the final effluent. It was shown in beaker experiments that chemical addition to humus tank effluent resulted in a better over-all phosphate removal than chemical addition to raw sewage due to the fact that all orthophosphate including that deriving from condensed phosphates could be precipitated.

As far as the removal of nitrogenous compounds is concerned, only biological removal appeared feasible. As biological denitrification requires carbonaceous energy in the form of COD, which is a limiting factor in biological filter effluents, the use of external carbon sources as energy supplement available at sewage treatment plants was investigated. Preliminary studies carried out at Daspoort, Pretoria, and two mass balance studies at Rooiwal, Pretoria, and Vlakplaats, Boksburg, showed that raw and digested sludges seemed to be the most promising carbon sources, as far as C:N ratio was concerned. Digested sludge, however, was considered not to be available in sufficient quantities, as its unbiodegrable fraction is higher than of raw sludge. Also, not every sewage treatment works has digested sludge at its disposal. Therefore, for pilot plant experiments carried out at

Daspoort and using biological filter effluent, raw sludge and raw and settled sewage were selected as the most suitable external carbon sources. It was shown that in a denitrification unit denitrification of humus tank effluent to values below 1,0 mg/ℓ NO_3-N was feasible. A denitrification rate of 17 mg NO_3-N per g VSS per h was achieved at a one-day sludge age. Settling problems observed in this study which caused loss of sludge in the effluent were overcome in full-scale experiments by chemical addition.

In addition to the information reported in this document, further information may be found in publications listed at the end of this report.

2. INTRODUCTION

During 1978 the National Institute for Water Research (NIWR) submitted a project proposal to the Water Research Commission (WRC) to investigate the chemical removal of phosphate from biological filter plants. It was realised that with more stringent phosphate requirements for sensitive areas in the near future that serious attention be given to conventional type work. A tripartite contract between the WRC, the NIWR and the Municipality of Pretoria was duly signed in 1979. Research carried out by the NIWR under this contract dealt with laboratory-scale experiments on chemical phosphate removal from biofilter effluents and with desk and pilot studies on the removal of nitrogenous compounds by biological means.

According to figures obtained from the Department of Health, Welfare and Pensions, 50 % of the number of all sewage treatment plants operated by municipalities in South Africa make use of biological filters, while 10 % use biological filters as well as activated sludge systems. Forty per cent use only the activated sludge process. Although not all of these biological filter plants are located in critical catchments, where phosphorus concentrations are limited by the effluent specifications promulgated in 1980, the necessity of upgrading these existing biological filter plants justifies research on the feasibility of nutrient removal in biological filter effluents.

Although up to now only orthophosphate is limited in specific areas to concentrations not exceeding 1 mg/ℓ (as P) in the effluent, stricter limits for nitrogenous components are expected in the near future. The only nitrogen limit generally in force allows a maximum ammonia concentration of 10 mg/ℓ (as N). However, moves are afoot to implement more stringent standards, which will drastically affect the operation of current systems.

Extensive research has been conducted in the field of nutrient removal, especially biological nitrogen and phosphate removal, by means of the modified activated sludge process. Although good results have been achieved and these are rapidly being implemented in new full-scale plants the problem of nutrient removal in the case of existing conventional sewage purification systems, including biological filters, still persists.

An important potential advantage of chemical nutrient removal is that the organic load on a biological filter can be decreased so that the hydraulic load may be increased. This is the major advantage in cases where sewage works already have reached their maximum capacity. An organic energy source and proper contact of sludge, nitrate-bearing effluent and substrate under anoxic conditions are prerequisites to biological nitrogen removal. It is also necessary to work within a well-defined temperature range to ensure optimum performance. The conditions required for biological phosphate removal include passage of the biomass through an anaerobic zone in the presence of sufficient concentrations of chemical oxygen demand (COD) followed by an aerobic stage, where the phosphate is taken up by the biomass or adsorbed onto the sludge. Since the metabolic processes in the latter zone leads to the formation of ammonia, which subsequently is oxidized to nitrate, an anoxic stage must be included in the system to achieve denitrification. For this purpose mixed liquor must be recycled from the aerobic zone to the anoxic zone (a-recycle).

The principles of pure biological nutrient removal have not yet been applied fully to biological filter systems. Therefore combined biological-chemical treatment is considered to be a most promising way of achieving nutrient removal from biological filter effluents.

In view of the above considerations, the Water Research Commission, the Municipalities of Pretoria and Boksburg and the National Institute for Water Research have entered into a quadripartite agreement which provides for research into ways and means of upgrading the performance of existing biological filter systems. Two substudies were formulated:

Substudy 1 provided for investigations on phosphate removal by means of $FeCl_3$-precipitation using existing full-scale biological filter systems. This study was carried out by the Municipality of Pretoria at Daspoort and by the Municipality of Boksburg at Vlakplaats, respectively.

Substudy 2 dealt with laboratory tests to develop chemical-biological methods to remove phosphate as well as nitrogen from biological filter effluents. These investigations were carried out by the National Institute for Water Research.

3. OBJECTIVES OF THE PROJECT AND MODUS OPERANDI

The aim of the research project was to develop and demonstrate practical means of upgrading or supplementing the performance of biological filter treatment works with respect to the following:

(i) Removal of phosphate in the effluent to levels of 1 mg/ (as P) or less.

(ii) Oxidizing residual nitrogenous compounds in the effluent, followed by biological removal of the reaction products to a total nitrogen concentration of 10 mg/ℓ (as N) or less.

(iii) Removal of COD from raw sewage to increase the treatment capability of biological filter systems which have reserve hydraulic capacity but which are organically overloaded.

The removal of carbonaceous material and phosphate can be achieved by precipitation, flocculation and coagulation with metal salts. To investigate the metal salt requirement for COD and phosphate removal jar tests were carried out using ferric chloride. These experiments involved tests on the effect of coagulant concentration on phosphate removal, the influence of coagulant mixing on phosphate precipitation and the effect of ferric sludge recycling.

The literature on the phosphate precipitation was reviewed in order to establish whether these findings could be applied to local sewage treatment works and whether they were in line with local results.

Colloidal nitrogenous compounds may be removed by coagulation, while ammonia may be removed, for example, by lime addition followed by air stripping or by precipitation as magnesium ammonium phosphate salts under carefully controlled conditions. Denitrification, however, has been shown to be feasible only when implemented biologically. The combination of a biological filter system and an activated sludge process was considered for this purpose. A minimum energy supply in the form of COD is required for such a process. Especially the C:N ratio, expressed as COD to TKN is considered to be very important. As most of the carbonaceous energy, however, is removed by passage through the biological filter or by preceding chemical treatment, a lack of energy for further biological treatment is likely to arise. For this reason the possibility of supplementing COD by means of external carbon sources was considered, particularly carbon sources available at sewage treatment plants such as digested sludge, raw sludge, digester supernatant, raw and settled sewage. The chemical composition and biodegradability of these sources were determined. Mass balances at two different sewage treatment works were carried out to estimate the available quantities of these carbon sources. Finally the use of the most promising energy supplements were tested under pilot plant conditions.

The research programme was reconsidered periodically on the basis of results available at that stage and amended when necessary at Technical Subcommittee and Steering Committee meetings held regularly throughout the three year tenure of the contract. It therefore happened that certain research tasks were dropped from the programme and more meaningful ones added. The overall programme, as approved after various interim amendments, has been condensed and tabulated for ease of reference (Table 3.1).

4. LITERATURE SURVEY ON BACKGROUND TO NUTRIENT REMOVAL IN BIOLOGICAL FILTER SYSTEMS

Much work has been carried out in the USA to investigate the upgrading of existing biological filter systems by either chemical treatment or design modifications. These studies included jar tests as well as pilot and full-scale investigations. Different coagulants were used and different ways of plant operation investigated. The plants considered in this literature review are summarized in Table 4.1. Relevant information is summarized and conclusions for practical application at local sewage works are drawn.

TABLE 3.1: Contract research programme (1979-1982)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
1979												
Laboratory studies on the flocculant requirements for phosphate precipitation				—	—	—	—	—	—	—	—	—
Literature survey	—	—	—	—	—	—	—	—	—	—	—	—
1980												
Laboratory studies on the flocculant requirements for phosphate precipitation	—	—	—									
Literature survey	—	—	—	—	—	—	—	—	—	—	—	—
Evaluation of degradability of external carbon sources				—	—	—	—	—	—	—	—	—
1981												
Evaluation of degradability of external carbon sources	—	—	—									
Mass balance at Rooiwal				—	—	—						
Pilot plant studies on denitrification						—	—	—	—	—	—	—
1982												
Pilot plant studies on denitrification	—	—	—	—	—							
Mass balance at Vlakplaats	—	—	—									

4.1 Phosphate species in sewage and the chemistry of phosphate removal

4.1.1 Phosphate species in sewage

If chemicals are added to remove phosphorus from sewage one must be aware of the fact that it is mostly orthophosphate which is precipitated at normal operating pH levels. It is therefore of importance to take cognisance of the various phosphate species in sewage. Three components are of particular interest in connection with chemical phosphorus removal: orthophosphate, condensed phosphate and total phosphate.

Orthophosphate (PO_4^{3-}) is defined as a chemical structure in which each phosphorus atom is tetrahidrally surrounded by four oxygen atoms. This orthophosphate group may occur as a triply charged anion (PO_4^{3-}) or as a structure in which there are covalent bonds from one or more of the four oxygen atoms to an atom other than a phosphorus atom, for example H_3PO_4, which exhibits an O-H bond, or acetyl phosphate $CO_3C(O)OPO_3H^-$, which shows a C-O bond (Van Wazer, 1973).

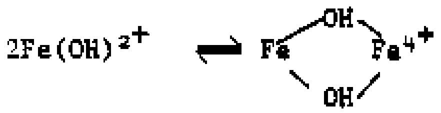
Condensed phosphates are formed by repeated condensation of tetrahedral PO_4^{3-} groups, which by linking through common oxygen atoms can share their corners with similar tetrahedra. In this way, highly polymerized one dimensional (chain), two-dimensional (sheet) or three-dimensional networks can be built up as well as smaller rings (Corbridge, 1974). Condensed phosphates occur in detergents and they are used as softeners due to their ability to sequester calcium ions and thereby reduce the effective water hardness by forming polyphosphate calcium complex ions (Van Wazer, 1973).

Total phosphate includes orthophosphate, condensed phosphate and organic phosphate. All these fractions can be determined together if converted to orthophosphate by digestion in the presence of strong acids (Standard Methods, 1975).

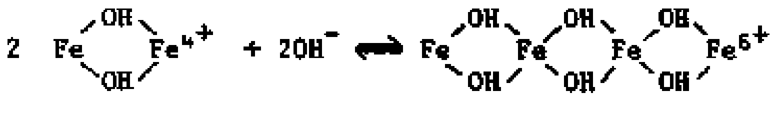
4.1.2 Chemistry of phosphate precipitation by iron salts

The most frequently used form of iron to precipitate is Fe(III)-salts. The chemistry of this salt in water is a fairly complex one and has been described in detail by Lijklema (1979). The following reactions will take place in water:

$$Fe^{3+} + H_2O \rightleftharpoons Fe(OH)^{2+} + H^+ \quad \ldots\ldots\ldots\ldots \quad (1)$$

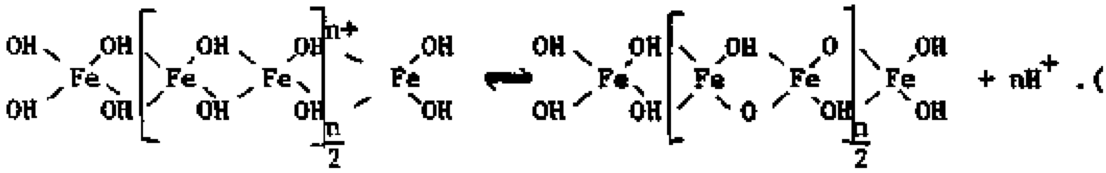

.... (2)

In the beginning there is not only the formation of iron hydroxide but also the start of polymerisation, which will carry on:


.... (3)

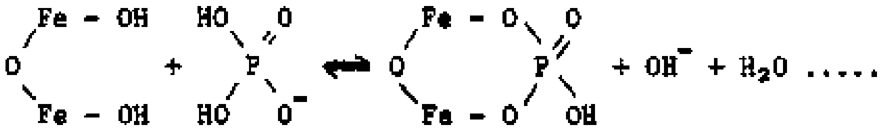
A

Due to further reactions with OH⁻ -ions these long chains (structure A) will increase according to structure B :


.(4)

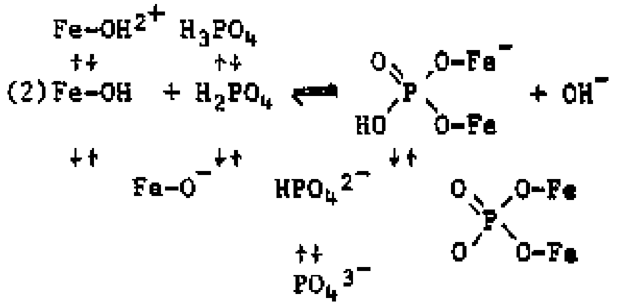
B C

This polymerization step is relatively quick, but there is also a slower reaction which converts structure B to structure C. This conversion can finally lead to the formation of goethite or haematite, where more and more of the OH⁻-bridges have been changed to O-bridges. It is known that adsorption of phosphate can take place on the surface of these metal oxides by forming bridges between phosphate ions and two hydroxyl groups at the surface according to equation (5):


..... (5)

It can be seen that the adsorption capacity depends very much on the number of free hydroxyl groups available. The fact that older solutions of iron hydroxide show less adsorption capacity than freshly precipitated solutions, can be explained by further polymerisation which decreases the number of binding sites. On the other hand if phosphates are in contact with hydroxides the adsorption capacity increases again. The most likely explanation is that changes in the structure and the length of the polymers cause this improvement. The decrease in charge accompanying the phosphate binding, favours the adsorption of protons and therefore a shift from structure C to B (equation 4). This results an increase in binding sites.

Very important is the molar ratio R of OH^- : PO_4^{3-} adsorbed. According to equation 5 R=1 is expected, but in practice different types of phosphates occur and the distribution of charges changes due to precipitation. Therefore the following equation is likely to be closer to reality:



.... (6)

The quotient R depends on the pH values as an increasing pH causes a decrease in positive charges of the hydroxide.

At pH values above 7, $FePO_4$ is believed to be the dominant reaction product. The Fe:P ratio (W/W) in $FePO_4$ is 1,8, but examples from practice show that a dosage of at least 10 mg/ℓ iron to the sewage is required for hydroxide formation (Culp and Culp, 1971).

4.2 Kinetic studies on phosphate precipitation using jar tests.

4.2.1 Experiments using synthetic phosphate solutions

Studying the kinetics and mechanism of phosphate precipitation with Fe(II), Fe(III) and Al(III) salts, Recht and Ghassemi (1970, 1971) used a synthetic phosphate solution which contained different forms of phosphate in concentrations simulating wastewater. The orthophosphate solution used contained 12 mg/ℓ P added as Na_2HPO_4 . The authors determined the kinetics of precipitation by means of a batch reactor studies with continuous monitoring of pH and conductivity. Five hundred mℓ quantities phosphate solution was stirred at 90 rpm while 5 mℓ of an aluminium or ferric nitrate solution was added to establish cation-to-phosphate molar ratios of 2:1, 1:1 and 0,5:1. After 2 min. of rapid mixing at 90 rpm, the stirring rate was reduced to 20 rpm and mixing was continued at this rate for 10 min., followed by a settling period of 20 min. They found that changes in pH and conductivity happened rapidly and occurred within 10 to 60 seconds after the addition of the precipitant, with no measurable change in pH or conductivity after this period. In one experiment at a 2:1 aluminium-to-orthophosphate molar ratio and a final pH of 5,9 in the solution it was found that a drop in phosphate concentration from the initial 12 mg/ℓ P to 0,10 mg/ℓ P occurred in less than 60 seconds after adding aluminium nitrate. After this period no further removal of phosphate was observed, monitoring up to 32 minutes, despite a very noticeable gradual growth and agglomeration of the precipitate flocs. Recht and Ghassemi (1970), in an experiment designed to give a finer resolution of reaction time, showed that the reaction of orthophosphate with both Al(III) and Fe(III) is completed in less than one second.

The removal of orthophosphate was found to be affected by both pH and concentration of added metal salts. If a 1:1 cation-to-orthophosphate molar ratio was used the residual phosphate concentrations were 1,85 mg/ℓ using iron and 3,5 mg/ℓ using aluminium. At a 2:1 cation-to-orthophosphate ratio the phosphate residuals were 0,07 mg/ℓ and 0,10 mg/ℓ with iron and aluminium salts respectively. The optimum pH was found to be close to 6,0 for Al(III) and in the order of 3,5-4,0 for Fe(III). Figure 4.1 shows that the optimum pH ranges were quite narrow. If the precipitation was carried out near the optimum pH the formation of large, settleable flocs was observed. Outside this pH range colloidal suspensions were formed which in some cases could only be removed by filtration. At higher as well as at very low pH levels no turbidity was observed after settling using aluminium.

However, at higher pH levels the iron phosphate solution, remained turbid after settling due to dispersion of ferric hydroxide flocs. This phenomenon was not observed at very low pH levels.

Recht and Ghassemi (1970) also tested to what extent condensed phosphate could be removed. They used a solution consisting of 18 mg/ℓ P added as tetrasodium pyrophosphate and 21,6 mg/ℓ added as sodium tripolyphosphate. The removal of these condensed phosphates by precipitation with aluminium as well as with iron salts was found to be strongly dependent on pH and on the concentration ratio of the coagulant. The optimum pH

TABLE 4.1: Sewage treatment plants investigated in the USA with regard to upgrading existing biofilter systems.

Plant	Modus of upgrading	Used for	Author
Tapia Park Treatment Plant, Las Virgenes Municipal Water District in Calabasas, California.	-	Seconary effluent from activated sludge plant used for phosphate precipitation studies.	Ghassemi and Recht, 1970, 1971.
Mason Farm Sewage Treatment Plant, Chapel Hill, North Carolina	Alum addition	Jar tests, pilot and full-scale experiments on lime, $FeC\ell_3$ and alum addition.	Brown and Little, 1977
Richardson Treatment Plant, Texas	Alum addition	Pilot and full-scale experiments on $FeC\ell_3$ and alum addition	Derrington, Stevens and Laughlin, 1973
City of Boulder, Colorado	Activated sludge system (planned)	Evaluation of the possibility to upgrade existing biofilter system.	EPA Report 908/5-78-003. 1978.
Fourteen plants in Michigan (no details given on locations).	$FeC\ell_3$ addition (11 plants) Alum addition (1 plant) Lime addition (1 plant) Hydrated lime addition (1 plant)	Full-scale operation	Pierce, 1978.
Six plants in Minnesota (no details given on locations).	Activated sludge system	Full-scale operation	Pierce, 1978.

levels were close to 4 and 5 for Fe(III) and Al(III), respectively. At the 2:1 cation-to-phosphate equivalence ratio the pyrophosphate residual was 0,9 mg/ℓ P when Fe(III) was added and 0,06 mg/ℓ P, using Al(III). For tripolyphosphate the residuals were 3,80 mg/ℓ P and 0,65 mg/ℓ P, respectively. Practically no phosphate was removed, if the pH deviated by ± 1 unit from the optimum level. The authors also tested the influence of ageing of dilute coagulant solutions on the orthophosphate precipitation. Fe(III) solutions were found to undergo hydrolysis which could be observed by a change in colour and also by rapid changes in pH and conductivity. The solution pH dropped from 3,00 to 2,58 during the first 24 hours and there was a steady drop of phosphate removal capacity.

On the other hand Al(III) was found to be stable over a period of two months. No changes in pH, conductivity and the capacity to precipitate phosphate were observed.

Ghassemi and Recht (1971) also reported on phosphate removal experiments carried out using Fe(II) . They found that dissolved oxygen had a great influence as it oxidised iron from the ferrous to the ferric state. The ferric ions are then hydrolysed subsequently. The pH of optimum orthophosphate removal is higher for Fe(II) than for Fe(III) or Al(III) and is in the vicinity of 8,0. At that pH the equivalence capacity of Fe(II) for phosphate removal is higher than that of Fe(III) and Al(III) . A molar ratio of 1 was found to be sufficient for removals down to 1,0 mg/ℓ . Although from these data Fe(II) seems to be more suitable for phosphate precipitation than Fe(III), there are several practical problems which arise from its use, which make it less suitable for general applications, as will be pointed out below.

4.2.2 Experiments using effluents from sewage treatment works

Ghassemi and Recht (1970, 1971) used secondary effluent from a small treatment plant at Calabasas, California for their investigations. The plant treats about 7 600 m^3/d of primarily domestic sewage. Sludge is digested aerobically with the supernatant liquor returned to the head of the process. The effluent used in the kinetic experiment contained 9 mg/ℓ orthophosphate (as P). A partial analysis of a filtered effluent sample showed a total phosphate concentration of 7,75 mg/ℓ P. The pH was about 7,3. More detailed information about

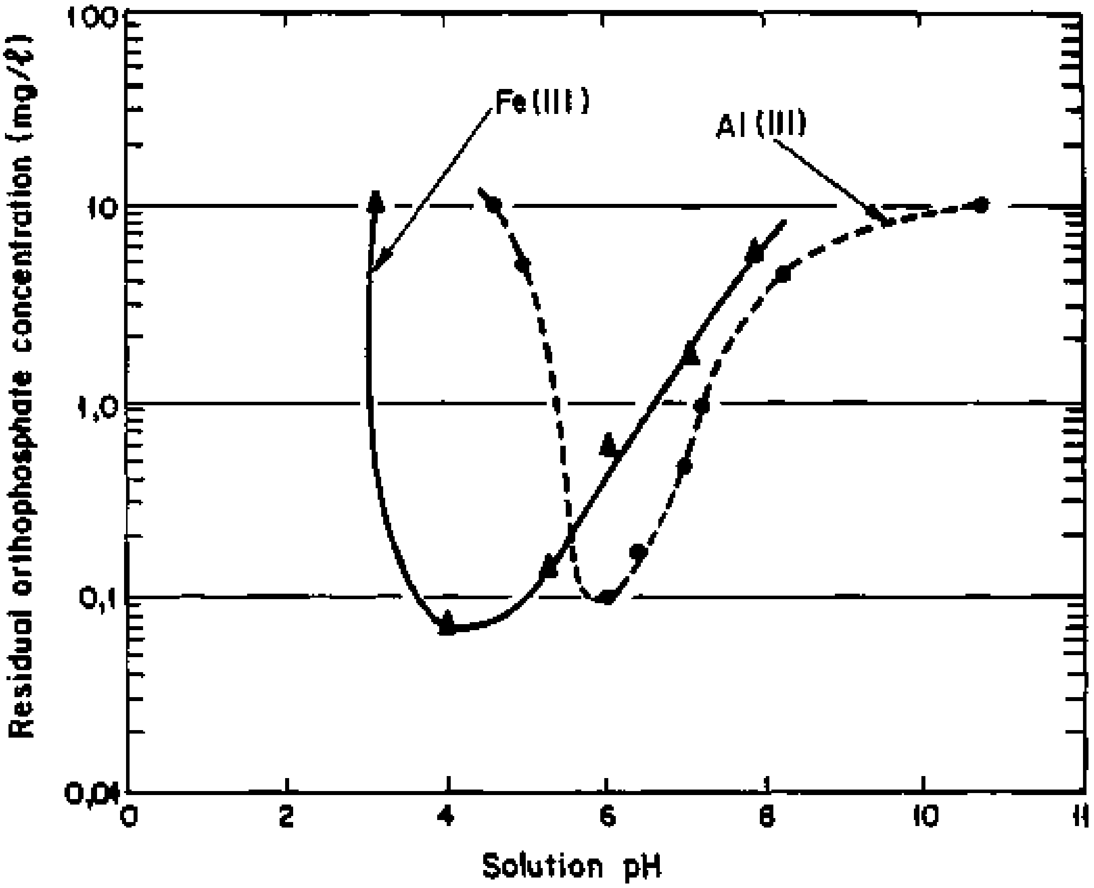


Figure 4.1: Precipitation of orthophosphate with Al(III) and Fe(III) at a 2:1 cation-to-orthophosphate molar ratio (initial orthophosphate concentration, 12 mg/ℓ P) (From Recht and Ghassemi, 1970).

the effluent was not available. They found that the removal of phosphate from secondary effluent is similar to the phosphate removal from pure solutions in that in both cases the efficiency of phosphate depends on solution pH.

Ghassemi and Recht (1971) also found that treating secondary effluent with Fe(II) in a molar ratio of 1:1 in the presence of dissolved oxygen yielded a very fine precipitate which did not settle well. Compared to the precipitates formed with Fe(III) and Al(III) the settling properties of the precipitate obtained with Fe(II) was inferior. The use of Fe(II) sources such as waste pickle liquor from steel mills may be an inexpensive source of Fe(II) to remove phosphate from wastewater under oxidising conditions and this salt may be used advantageously under such conditions.

Jar tests carried out by Brown and Little (1977) at Chapel Hill, North Carolina, evaluated also the use of lime besides ferric chloride and alum. The effluent was obtained from a high-rate biological filter plant treating relatively soft water. The alkalinity was about 140 mg/ℓ (as $CaCO_3$) and the total phosphate concentrations 10 mg/ℓ (as P). The diurnal orthophosphate loading rate ranged from 9 to 112 kg per day in the influent. The authors found that lime dosage up to 300 to 400 mg/ℓ (as $Ca(OH)_2$) at a pH of 11 or greater yielded removal of total inorganic phosphorus to a residual of less than 1 mg/ℓ (as P). A pH of 11 was found to be optimal, because at pH levels less than 11, lime reacted with soluble phosphorus to form a finely divided insoluble material, much of which did not settle out during the jar tests. The addition of a cationic polyelectrolyte, however, enhanced coagulation and settling properties at a pH of 9,3 to 9,5 and gave total inorganic phosphorus removal to residuals less than 1,0 mg/ℓ P at a lime dosage of about 80 mg/ℓ .

Investigating the effectiveness of alum dosage Brown and Little (1977) used plant influent and secondary effluent. The influent had an average BOD_5 concentration of 154 mg/ℓ, while the total phosphate and TKN concentrations were 10,4 and 31,2 mg/ℓ respectively. The secondary effluent had a total phosphate concentration of 10,5 mg/ℓ . They found that, to achieve a 97 to 98 per cent removal of total phosphate an alum dosage of 200 mg/ℓ as $Al_2(SO_4)_3.18H_2O$ was required using plant influent, whereas 150 mg/ℓ alum was required to treat secondary effluent. With regard to the Al-to-total-phosphate ratio, however, they found that a molar ratio between 1 and 2 is required to effect the same removal, regardless of whether plant influent or secondary effluent is used. Brown and Little (1977) used two different sources of Fe(III) ions in their jar tests. They found that lesser concentrations of ferric chloride was required to remove phosphates, solids and turbidity than of ferric sulphate. On the other hand, precipitation with ferric sulphate yielded less sludge and stronger flocs. With both ferric chloride and ferric sulphate the phenomenon of iron-leakage, which means iron going back in solution and causing brownish colour of the effluent, was observed. An experiment was carried out to determine the optimum pH for both removal of phosphate and elimination of effluent iron. Brown and Little (1977) found, that with Fe(III) dosed at 50 mg/ℓ phosphate removal was the same over the pH range 6 to 8, but iron capture was enhanced by increasing pH to levels greater than 7 by adding lime. The authors concluded that the 'iron-leakage' observed in practice is most probably due to the escape of colloidal iron particles which do not settle under normal operating conditions. Using biological filter effluent from the Chapel Hill treatment plant approximately 90 mg/ℓ $Ca(OH)_2$ was required to raise the pH above 7 after dosing 50 mg/ℓ Fe(III) . They also found that if lime was used for pH control ferric sulphate was superior to alum for phosphate removal, but less effective than ferric chloride.

4.3 Phosphate removal studies at pilot and full scale

4.3.1 Pilot plant studies

Brown and Little (1977) used for their experiments at Chapel Hill a pilot plant which received its wastewater from the main plant after passage through a bar rack, degritting chamber and a fine bar rack. The pilot plant consisted of a primary settling tank, a biological filter, and a final settling tank. Recirculation of biological filter effluent to the head of works was provided around the filter through the primary settling tank, using a 2:1 recirculation ratio. The influent characteristics were as given above in 4.1.2. The authors added alum in concentrations of 100, 150 and 200 mg/ to the primary clarifier influent and secondary clarifier influent ahead of recirculation.

There was an increase in the total suspended solids (TSS) concentration in the effluents from the units where alum had been added, which is contradictory to the results from the jar tests and the authors, therefore, concluded, that under the conditions of this pilot plant study TSS removal was not enhanced by alum precipitation. They rather found by visual observation, that the character of effluent TSS during alum addition differed from that when no alum was added, in that alum flocs were visible during alum addition and most probably contributed to the effluent TSS . Alum addition to the primary clarifier was found to be more effective than addition to the biological filter and as good or better than split-treatment to the primary and secondary clarifiers. Alum addition to biological filter influent or to biological filter effluent above the take-off point of recirculation was less effective for phosphorus removal than addition to primary clarifier influent or to influent to the secondary clarifier below the take-off point or recirculation.

4.3.2 Full-scale studies

The results obtained from pilot plant studies by Brown and Little (1977) were applied on a full-scale plant and evaluated further. They added alum in a molar ratio of 2:1 (Al:P) and returned settled sludge from the final clarifiers to the primary clarifiers. As a parameter of interest they examined the diurnal characteristics of the biological filter effluent entering the final clarifier. They tested for total phosphate, because it was thought to be most significant in terms of alum requirements. During the sampling period the recirculation of biological filter effluent to raw sewage was held at a ratio of 2:1 and this recirculation was found to suppress almost completely the diurnal variation in BOD, TSS and total phosphate. The dosage of alum was between 175 and 220 mg/ℓ, which is equal to an Al: influent total phosphate mole ratio of 1,7 to 2,2. It was found, that when this ratio approaches 1,0 phosphate was precipitated but not effectively settled. The total phosphate removal averaged 82 per cent using alum treatment compared to an 18 per cent removal in the control unit without chemical addition. The average total phosphate concentration in the influent was 12 mg/ℓ and that in the effluent was about 2,2 mg/ℓ in the unit treated with alum and about 9,7 mg/ℓ in the control unit. The concentrations of soluble phosphate in the effluent was 0,45 mg/ℓ and 6,7 mg/ℓ respectively. In the case of BOD_5 an average removal of 91 per cent was achieved compared to 77 per cent without chemical addition, which is equal to average effluent concentrations of 15 mg/ℓ and 40 mg/ℓ respectively. Total suspended solids was removed by 88 per cent and 77 per cent respectively, which yielded effluent concentrations of 28 and 56 mg/ℓ, respectively.

Brown and Little (1977) found that the return of alum sludge had a significant effect on the performance of the primary clarifiers to which the sludge was pumped. The total phosphate concentration in the primary clarifier receiving alum sludge was reduced from 11,6 mg/ℓ to 6,5 mg/ℓ which is equal to a removal of 44 per cent. The control unit without chemical treatment only showed a reduction to 10,0 mg/ℓ P, equal to a removal of 14 per cent. There was also an improvement of BOD_5 and total suspended solids removal due to alum sludge recirculation.

Darrington, Stevens and Laughlin (1973) investigated the effectiveness of alum and ferric chloride addition at Richardson sewage works, Texas. This plant is a typical standard-rate single-stage biological filter system, treating domestic sewage. The influent BOD_5 concentration was 166 mg/ℓ and the total phosphate concentration was 11,0 mg/ℓ on average.

Liquid alum was fed at points respectively just before the final clarifier, before the primary clarifier and in a split-treatment mode. Chemical treatment preceding the final clarifier yielded an effluent phosphate concentration of 2,0 mg/ℓ equalling a removal of 81 per cent. The suspended solids concentration was reduced by 92 per cent resulting in 13 mg/ℓ in the effluent and BOD_5 was removed by 95 per cent yielding 6,2 mg/ℓ residual.

When the chemical dosage was changed to a point preceding the primary clarifier the performance of the plant deteriorated: phosphate was removed by 77 per cent, giving a residual of 2,6 mg/ℓ, BOD_5 was reduced by 80 per cent to a residual of 27 mg/ℓ and the suspended solids concentration in the effluent was 14 mg/ℓ (91 per cent removal). The primary effluent was described as murky or turbid most of the time and the characteristic persisted all the way through the final clarifier. Due to technical problems turbidimetric results were not available.

Two different ways of split-feeding were tried. Firstly, 80 per cent of alum was dosed to the final clarifier and 20 per cent to the primary clarifier. At an Al:P mole ratio of near 2:1 phosphate was reduced to a residual of 0,3 mg/ℓ in the effluent, equal to a removal of 96 per cent. Sludge recirculation of $^1/_{500}$ of the influent was maintained. When a split-feeding of 70 per cent to the final clarifier and 30 per cent to the primary clarifier was tried, the results were not as good as when a higher percentage was dosed to the final clarifier. After eighteen days the plant started to develop problems similar to those which had been found when all the chemicals were fed to the raw influent.

For longer lasting experiments it was therefore decided to only dose ahead of the final clarifier. Over a period of 11½ months a phosphate removal of 96 per cent, BOD_5 removal of 97 per cent and a removal of suspended solids of 95 percent was obtained yielding residuals of 0,49 mg/ℓ P, 4,5 mg/ℓ and 7,0 mg/ℓ, respectively. The average Al:P mole ratio was 1,6.

For comparison liquid ferric chloride was added in a similar way as alum. The Fe:P mole ratio was 1,9. When the chemical was added prior to the final clarifier phosphate was reduced by 80 per cent, which gave an effluent concentration of 2,4 mg/ℓ . Suspended solids and BOD_5 were removed by 86 per cent, resulting in effluent residuals of 26 mg/ℓ and 28 mg/ℓ, respectively. These effluent concentrations were higher than without chemical treatment. When ferric chloride was added prior to the primary clarifier phosphate was removed by 85 per cent (residual: 1,8 mg/ℓ), BOD_5 was reduced by 79 %, yielding 38 mg/ℓ residual and suspended solids removal was 86 per cent, which gave 23 mg/ℓ residual and suspended solids removal was 86 per cent, which gave 23 mg/ℓ residual.

The phenomenon of iron leakage was observed in both cases, but considerably more iron was found in the effluent, when ferric chloride was added prior to the final clarifier than when it was dosed ahead of the primary clarifier. The addition of polyelectrolyte to the final clarifier reduced iron leakage by settling colloidal iron.

4.4 Problems in plant operation during chemical treatment

4.4.1 Sludge production

Chemical treatment results in considerably more sludge (by volume) than conventional sewage treatment. This is of importance when considering whether sufficient digester capacity is available to handle the additional amount of sludge. Brown and Little (1977) reported that in the case of alum treatment the total volume of sludge removed from the unit was 32 per cent greater than from the unit without chemical addition. In the unit with alum addition, 6,28 ℓ/m^3 was pumped to the digesters compared to 4,75 ℓ/m^3 in the unit without chemical addition. The total solids was 3,57 per cent in the case of chemical treatment, which gave an absolute figure of 107,28 g/m^3 . The volatile fraction was 67 per cent or 151,8 g/m^3 . The sludge from the unit which did not receive any alum showed a percentage of total solids of 3,85, 177,96 g/m^3 solids were pumped and the volatile fraction was 76 per cent, which gives an amount of 134,4 g/m^3 solids pumped. This means that in the case of chemical treatment the amount of total solids which is withdrawn is 28 per cent higher and the amount of volatile solids withdrawal is 13 per cent higher than for the unit without alum dosage. Derrington *et al.* (1973) found that if alum was added at a point preceding the primary clarifier at the Richardson plant, Texas, 50 percent more primary sludge was produced than when the plant was run in a conventional mode without any chemical addition. Experiences from biofilter plants in USA using chemical addition for upgrading showed that, although chemical treatment increased the solids loading on sludge digester, no additional or modified solids processing facilities were needed. Increased quantities of digester supernatant was treated with the raw sewage and no particular difficulties were observed.

As all these full-scale studies had been carried out using alum addition, no results were reported on the use of iron-salts.

4.4.2 Digester problems

In general no or only minor problems are reported from digesting sludge containing metal salts and polymers. Derrington *et al.* (1973) observed problems in sludge digesting at Richardson only when alum was added preceding the primary clarifiers. Stratification was observed within the digester which resulted in two layers of solids, one heavy large layer on the bottom and one fresh light layer at the top. Both layers were separated by a layer of relatively clear supernatant. This situation developed within one week. The same problem occurred when split-feeding was tested at a percentage of 70 per cent to the final clarifier and 30 per cent to the primary clarifier, but not when the relation was 80/20 per cent. The authors also noticed a drop in the concentration of digested sludge from 8 to 6 per cent during alum addition. The digested sludge was found to dewater quite rapidly. In the Richardson study the drying time of 22 days was reduced from 8 to 10 days. Iron sludge was found to dry even quicker. Brown and Little (1977) described a drop in pH in the digester due to the alum treatment, but this problem could be overcome by adding lime. The digestion process itself, however, continued to produce a normal reduction in volatile solids throughout the whole period of alum treatment. Another problem arose from the supernatant from the secondary digester. The total suspended solids concentration increased from normal less than 1000 mg/ℓ, to 8000 to 20 000 mg/ℓ . The sludge in the secondary digester did not thicken in a normal manner. This 'heavy' supernatant, due to the higher concentration of suspended solids, especially, caused problems as the supernatant return line tended to clog. On several occasions 'heavy' supernatant had to be discharged to the undosed units, which caused upsets in that system. No problems were reported concerning dewatering of digested alum sludge.

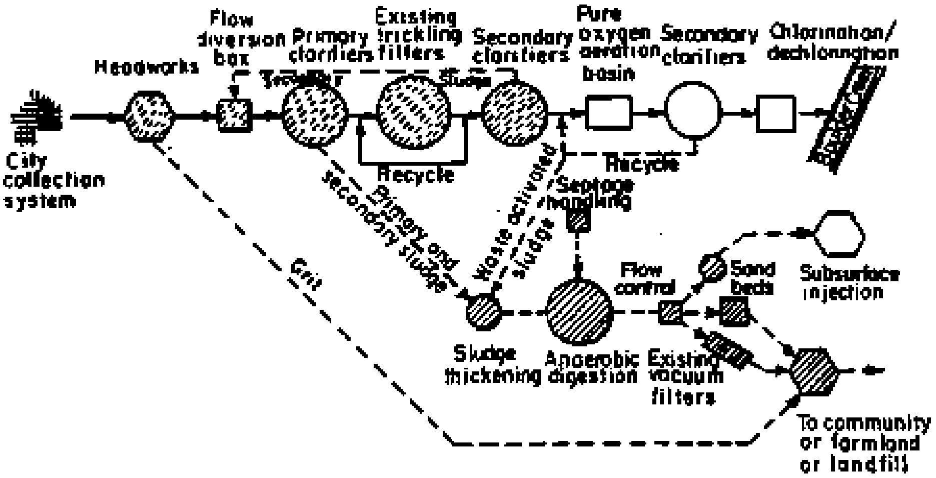
4.5 The use of activated sludge units in conjunction with biological filter systems.

The combination of biological filter systems and activated sludge plants is already in use in the USA and more of these units are planned. Pierce (1978) reported on 6 plants in Minnesota which use activated sludge units following a biological filter system. Different flow patterns are used at these plants. At one plant the flow from the primary sedimentation tanks is divided in 2 equal parts: one half is pumped to a biological filter, the other half to the activated sludge reactor. The effluent from the settling tank following the biological filter mixes with the primary effluent going to the aeration tanks. The filter effluent and the activated sludge mixed liquor are returned to the raw sewage well and recycled to the primary sedimentation tanks with the raw wastes. The load on the activated sludge systems is about 1,2 kg BOD_5/m^3 (aerator volume) and the average detention time including return activated sludge is 4 hours. Both the raw BOD_5 and suspended solids are reduced by 96 per cent yielding 11 mg/ℓ and 7 mg/ℓ, respectively, in the effluent.

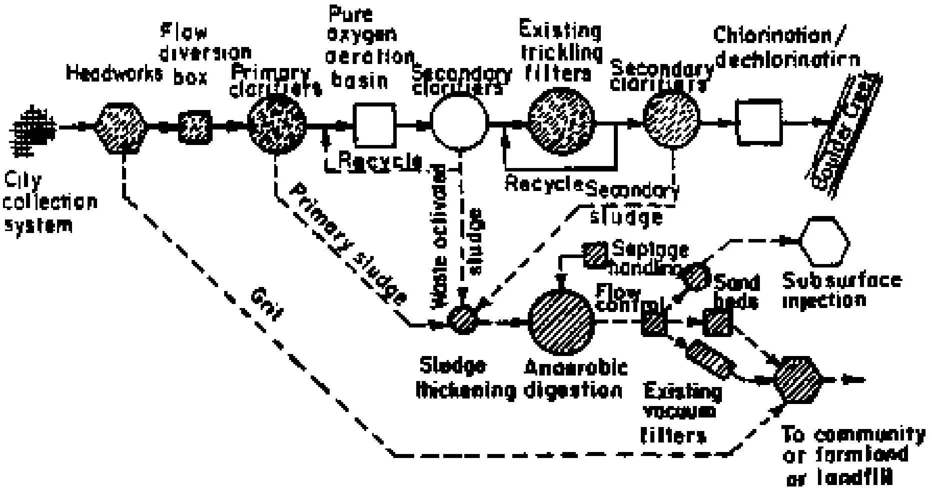
At another plant at Minnesota the biological filter effluent is discharged directly to the activated sludge process without intermediate settling. Suspended solids and BOD_5 were reduced by 94 and 96 per cent in the whole system, which gave effluent concentrations of 28 mg/ℓ and 24 mg/ℓ, respectively.

The other 4 plants which were mentioned showed results in the same order. Unfortunately no figures are available concerning nitrogen removal or the strength of the biological filter effluent entering the aeration tank.

Preliminary studies have been carried out by the U.S. Environmental Protection Agency, Region VIII, Denver, Colorado at Boulder, Colorado (1978), to upgrade an existing biological filter to meet effluent standards as this plant is in a sensitive area and discharges in the Boulder creek, which is part of a nature reserve. The addition of a pure oxygen activated sludge unit had also been considered amongst other possibilities, such as chemical treatment, infiltration ponds, polishing ponds, high-rate irrigation and multi-media filtration. Two different configurations were tested: an activated sludge unit prior to the trickling filter and an activated sludge unit following the biological filter, respectively. The activated sludge units consisted of an on-site oxygen generation plant, aeration basins and additional secondary clarifiers. The oxygen-activated sludge was designed to produce an estimated 90 per cent reduction in organic pollutants, in the case where it is located prior to the biological filter. The effluent from this unit would then be treated in existing biological



Alternative system A
Existing trickling filter plant plus Phase I additions and activated sludge process



Alternative system B
Activated sludge process ahead of existing trickling filter plant plus dechlorination

Figure 4.2: Possible upgrading of Boulder sewage plant by additional activated sludge systems (EPA Report 908/5-78-003, 1978).

filters, where ammonia-nitrogen could be converted to nitrate-nitrogen. In the case of the activated sludge unit following the trickling filters the activated sludge would oxidize any biodegradable organic matter leaving the biological filters and convert ammonia-nitrogen to nitrate-nitrogen. Both possible configurations are shown in Figure 4.2. Preliminary estimates of treatment effectiveness are summarized in Table 4.3.

Unfortunately no data concerning phosphorus removal or denitrification are available from this report. Both configurations were considered superior to other upgrading facilities such as infiltration ponds, polishing ponds and high-rate irrigation of effluent, but the activated sludge unit following the trickling filter was favoured by the Municipality of Boulder.

TABLE 4.2: Preliminary estimates of treatment effectiveness. Activated sludge unit prior to biological filter (A). versus activated sludge unit after biological filter (B). (From EPA report 90805.78-003, 1978)

	Suspended solids (mg/ℓ)	BOD$_5$ (mg/ℓ)	NH$_3$-N (mg/ℓ)
Raw wastewater estimate	140	147	30
Colorado monthly effluent standards	30	30	4,3 (weekly)
Boulder monthly effluent standards	20	20	4,3 (weekly)
A	15	10-15	<1
B	10	10	<1

4.6 Conclusions regarding chemical addition in conjunction with biological filter treatment.

The following conclusions are drawn from the literature study:

4.6.1 Alum and ferric chloride are the most frequently used chemicals to remove phosphorus from biological filter effluents to levels of approximately 1-2 mg/ℓ . In most of the cases alum is favoured. In those cases where ferric chloride was used, it was not clear if it was preferred as a result of superior phosphate removal capacity or due to economical considerations. The choice as to which coagulant is used also depends on the type of sewage, as both coagulants showed different pH optima. For ferric chloride the optimum pH is at values of about 4 while for alum it is about 6 at a molar ratio of 2:1 (cation-to-orthophosphate).

4.6.2 Polyelectrolytes are often required to improve settleability of chemical sludge and to prevent 'iron leakage' in the case of ferric chloride dosage.

4.6.3 Recirculation of biological filter effluent to be head of works was found to almost completely suppress the diurnal variation in BOD_5, TSS and total phosphate in the raw sewage. The return of alum sludge to the primary clarifier was found to reduce total phosphate, BOD_5 and TSS in the primary clarifier.

4.6.4 Dosage points of metal salts both before or after the trickling filter have been studied. Polyelectrolytes are added mostly before the final clarifier. In the few cases where lime has been used the dosage point is ahead of the final clarifier.

4.6.5 Split-feeding of coagulant was found to yield a slight improvement in phosphate removal but the result did not justify practical application.

4.6.6 Adding chemicals caused a 30 to 50 % increase in sludge volume to be treated in digester, but undigested sludge handling properties were not adversely affected.

4.6.7 Both alum and iron sludge could be dewatered easily after digestion, mostly in shorter times than normal digested sludge.

4.6.8 Some digester problems, such as a drop in pH, stratification, forming sludge blankets in the digester and an increase in suspended solids concentrations of the digester supernatant, which caused clogging op pipelines, have been reported. These however seem to have been caused rather by operational misjudgements than metal salt dosage. Drops in pH could be overcome by adding lime, but these pH changes did not affect the digester performance as far as gas production was concerned.

5. LABORATORY TESTS ON THE METAL SALTS REQUIREMENTS FOR COD AND PHOSPHATE REMOVAL.

5.1. Introduction

It is generally agreed that chemical treatment is the only feasible method to remove phosphates from biological filter effluents. The choice of coagulant is beside economical consideration dependent on the type of wastewater to be treated. Parameters like contact time of coagulant and wastewater and stirring intensity are also to be investigated on laboratory scale. In order to determine the parameters necessary for full-scale application of chemical phosphate removal extensive jar tests were carried out. Ferric chloride was chosen as coagulant. Experiments using different concentrations of ferric chloride and different stirrer speeds were carried out, in order to establish the optimum conditions for phosphate precipitation. The effect of chemical addition on the removal of nitrogenous and carbonaceous components in wastewater was also investigated. A 24-hour phosphate profile of Daspoort domestic wastewater was carried out to study the diurnal change in the concentration of different phosphate species with special regard to condensed phosphates. The effect of prefilter and post-filter treatment was studied and the effect of condensed phosphate concentrations in the raw sewage was investigated.

5.2. Materials and Methods

5.2.1 Sewage samples

Screened raw sewage of mainly domestic origin from Pretoria West, Pretoria Central and some southern suburbs such as Brooklyn and Hatfield, was sampled at 09h00, 11h00, 13h00, 15h00 and 17h00 at Daspoort sewage works. The average transit time in the sewers is approximately four hours (Thirion, personal communication).

5.2.2 Jar tests

Samples of 500 mℓ screened raw sewage each were vigorously stirred (80 rpm) for 2 min using a Phipps and Bird stirrer, immediately after the simultaneous addition of different concentrations of $FeCl_3$. This was followed by gentle stirring for 8 min at 20 rpm followed by a 10 min settling period after which samples were taken from the supernatant and chemically analysed.

The ferric chloride used in these tests was made up as a stock solution, containing 20 mg of 43 per cent (w/w) $FeCl_3$ in 2 mℓ distilled water.

5.2.3 Chemical analyses

Chemical oxygen demand (COD), TKN, ammonia (NH_4^+), total phosphate, orthophosphate and condensed phosphate were determined manually according to the Analytical Guide (1974). Nitrite-nitrate (NO_2^--NO_3^-) was measured using the Technicon Autoanalyzer technique.

5.3 Results

5.3.1 Phosphorus profile over 24 hours

In order to measure the diurnal variations in the phosphorus content of the sewage a 24-hour profile was taken. The parameters total phosphate, condensed phosphate, orthophosphate and COD were determined using samples at hourly intervals. The con- densed phosphate determination had been carried out immediately after sampling. The results are shown in Figure 5.1 and listed in Table 5.1.

The orthophosphate content does not vary much during the day and remained more or less constant at 4 mg/ℓ as P. On the other hand, large differences in total phosphate and condensed phosphate concentrations were observed. The condensed phosphate content showed a maximum at about 10h00 to 16h00 and a minimum of near zero between 18h00 and 06h00. This variation in the condensed phosphate content reflects daily life practices. Most of the condensed phosphate in domestic sewage originates from detergents and soaps mainly in use during the morning hours.

The total phosphate content exhibited peaks at 11h00 and 15h00. Afterwards the values decreased, reached another maximum at 02h00 and showed a minimum at 04h00 after which the amount of total phosphate increased again. The flow rate of the influent sewage showed a diurnal change pattern similar to that of the change in phosphorus composition: There was a maximum at 12h00 and a minimum during the night hours. Figure 5.1 illustrates that the phosphate was more efficiently removed

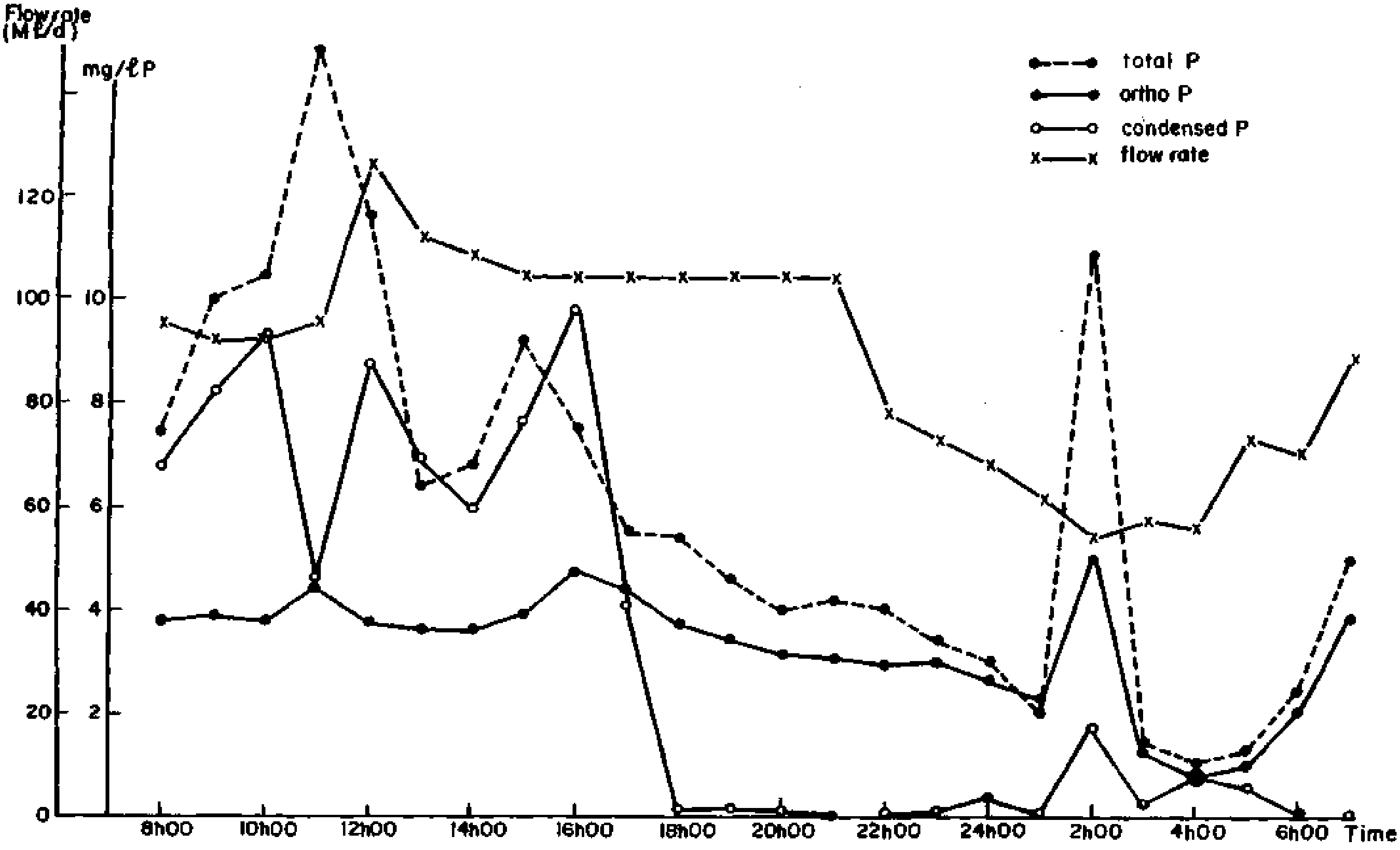


Figure 5.1: Diurnal changes in the concentration of phosphate species in Daspoort raw sewage.

TABLE 5.1: Diurnal changes in the concentration of phosphate species in Daspoort raw sewage

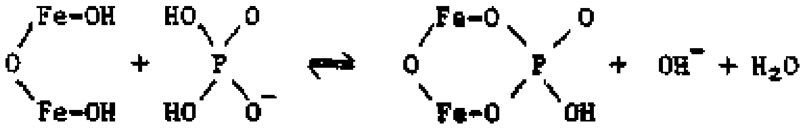
Sampling time	Orthophosphate (mg/ℓ as P)	total phosphate (mg/ℓ as P)	condensed phosphate (mg/ℓ as P)	COD (mg/ℓ)
08h00	3,7	7,4	6,74	485
09h00	3,8	10,0	8,17*	636
10h00	3,7	10,4	9,24*	551
11h00	4,4	14,8	4,53	1048
12h00	3,7	11,6	8,74*	444
13h00	3,6	6,4	6,88*	435
14h00	3,6	6,8	5,88*	470
15h00	3,9	9,2	7,59*	556
16h00	4,7	7,0	9,75*	746
17h00	4,4	5,5	4,10*	-
18h00	3,6	5,4	0,06	443
19h00	3,4	4,6	0,10	422
20h00	3,1	4,0	0,09	314
21h00	3,0	4,2	0,00	243
22h00	2,9	4,0	0,00	213
23h00	3,0	3,4	0,05	266
24h00	2,6	3,0	0,34	181
01h00	2,1	2,0	0,00	81
02h00	5,0	10,8	1,72	718
03h00	1,2	1,4	0,22	63
04h00	0,7	1,0	0,75	39
05h00	0,9	1,2	0,51	34
06h00	2,0	2,4	0,00	84
07h00	3,8	5,0	0,00	264

*Discrepancies due to analytical error

precipitation with $FeCl_3$ at times when the condensed phosphate concentration in the sewage was lower.

5.3.2 Studies on the mechanism of phosphate precipitation

Figure 5.2 shows the relationship between phosphate removal from Daspoort settled sewage and the applied Fe:P molar ratio under the conditions of jar tests. The precipitation of phosphate by iron salts is not stoichiometric, with Fe:P ratios of 2 or more being required for total phosphate removal to residual levels lower than 2 mg/ℓ (as P). This is ascribed to the conformation of ferric hydroxide, which, however, may adsorp phosphates according to the following equation (Lijklema, 1979):



The adsorption is favoured by thorough mixing of coagulant and the wastewater to be treated. The influence of stirring on the phosphate remaining capacity at different $FeCl_3$-concentrations is shown in Table 5.2 and Figure 5.3. This experiment refers to the second phase in the stirring experiment, the slow stirring to allow flocculation. The results show that stirrer speeds between 20 and 30 rpm were the most favourable for successful phosphate precipitation. At higher speeds the floc might be disrupted again. A lower speed also did not produce the result desired.

As for a particular water and type of coagulant applied the optimum rapid mixing time is dependent on the velocity gradient and the coagulant dose applied the root mean square velocity gradient (G) as defined by Camp (1946) was calculated for the experimental jar tests systems:

$$G = (W/\mu)^{\frac{1}{2}}$$

where G = average velocity gradient (s^{-1})
μ = absolute viscosity of the flocculating regime (Ns/m^2)
= 10^{-3} Ns/m^2 for water at 20_2°C
W = dissipation function (N/(m .s))
= total power dissipated per unit volume of chamber
= mean shear per unit volume per unit line.

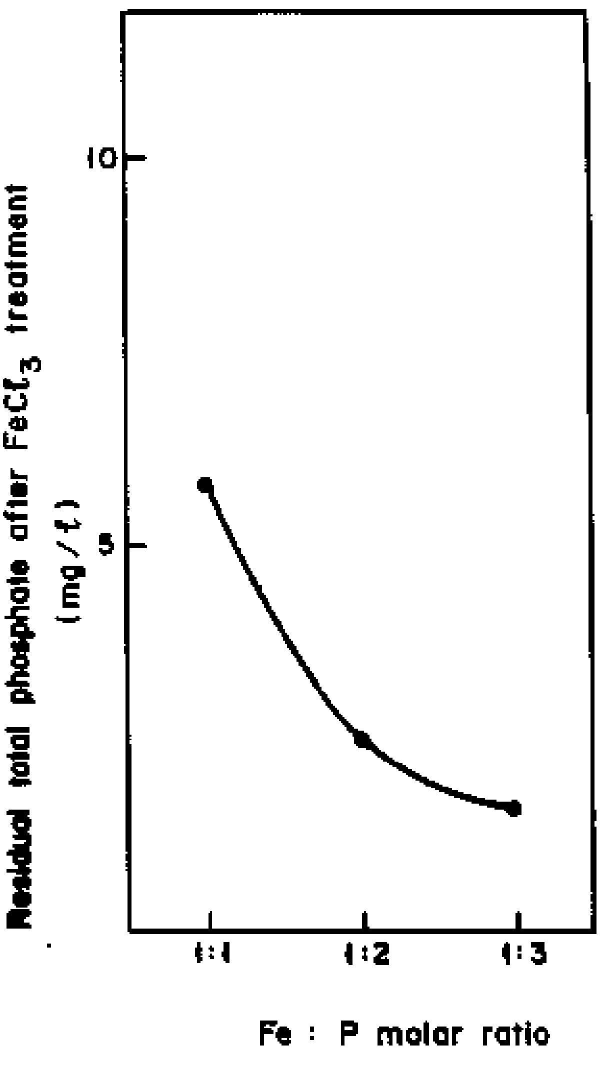


Figure 5.2: Relacionship between phosphate removal and Fe:P ratio

In Figure 5.4 the relationship between stirring speed and velocity gradient is shown. The best precipitation of orthophosphate by $FeCl_3$ was therefore achieved at a mean velocity gradient G between 19,5 sec^{-1} and 37,5 sec^{-1}, corresponding to a stirrer speed between 20 and 30 rpm. These results are in line with findings by Culp and Culp (1971), who reported of velocity gradients commonly used of 9 to 30 sec^{-1}.

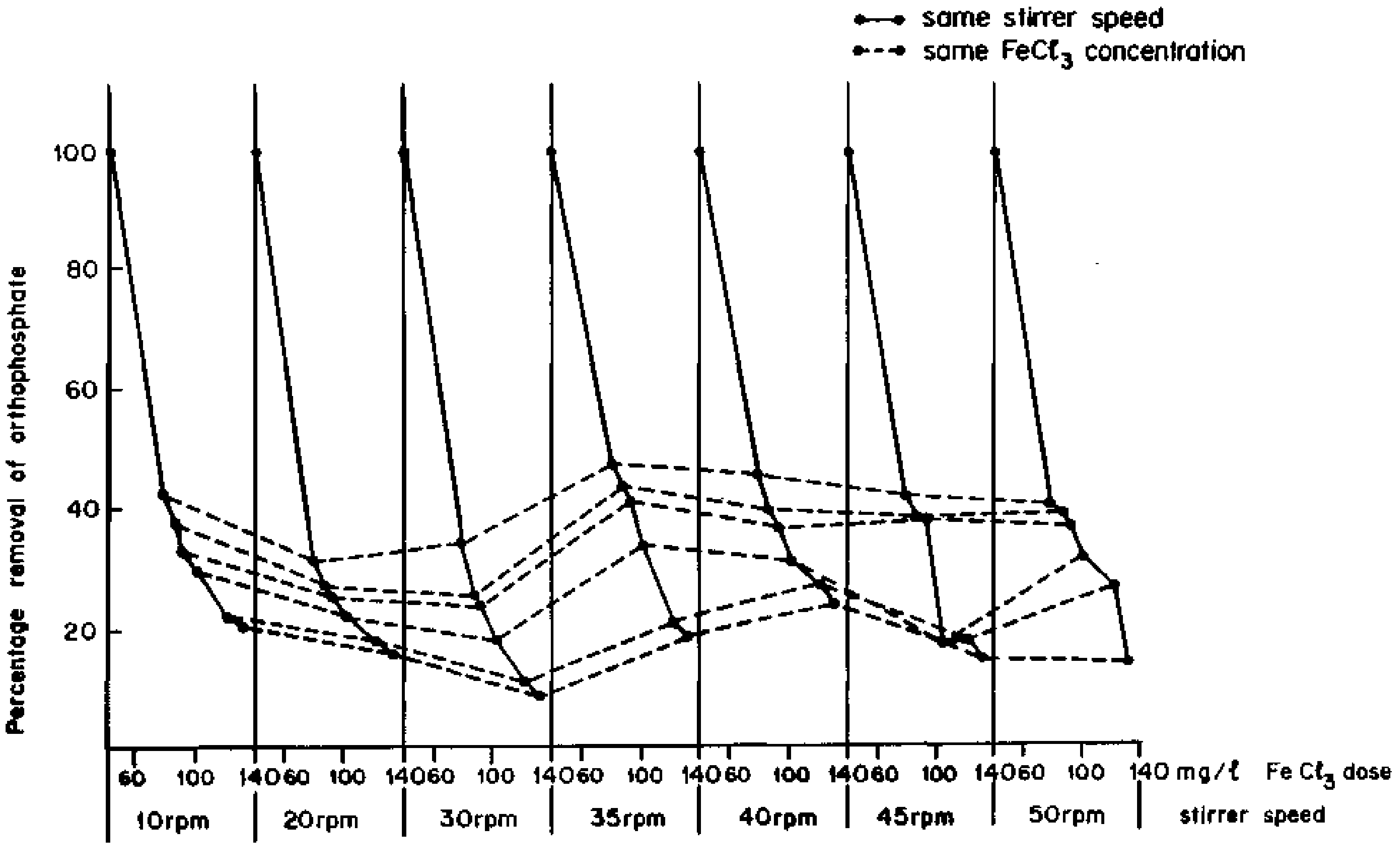


Figure 5.3: Influence of various stirrer speeds on flocculation

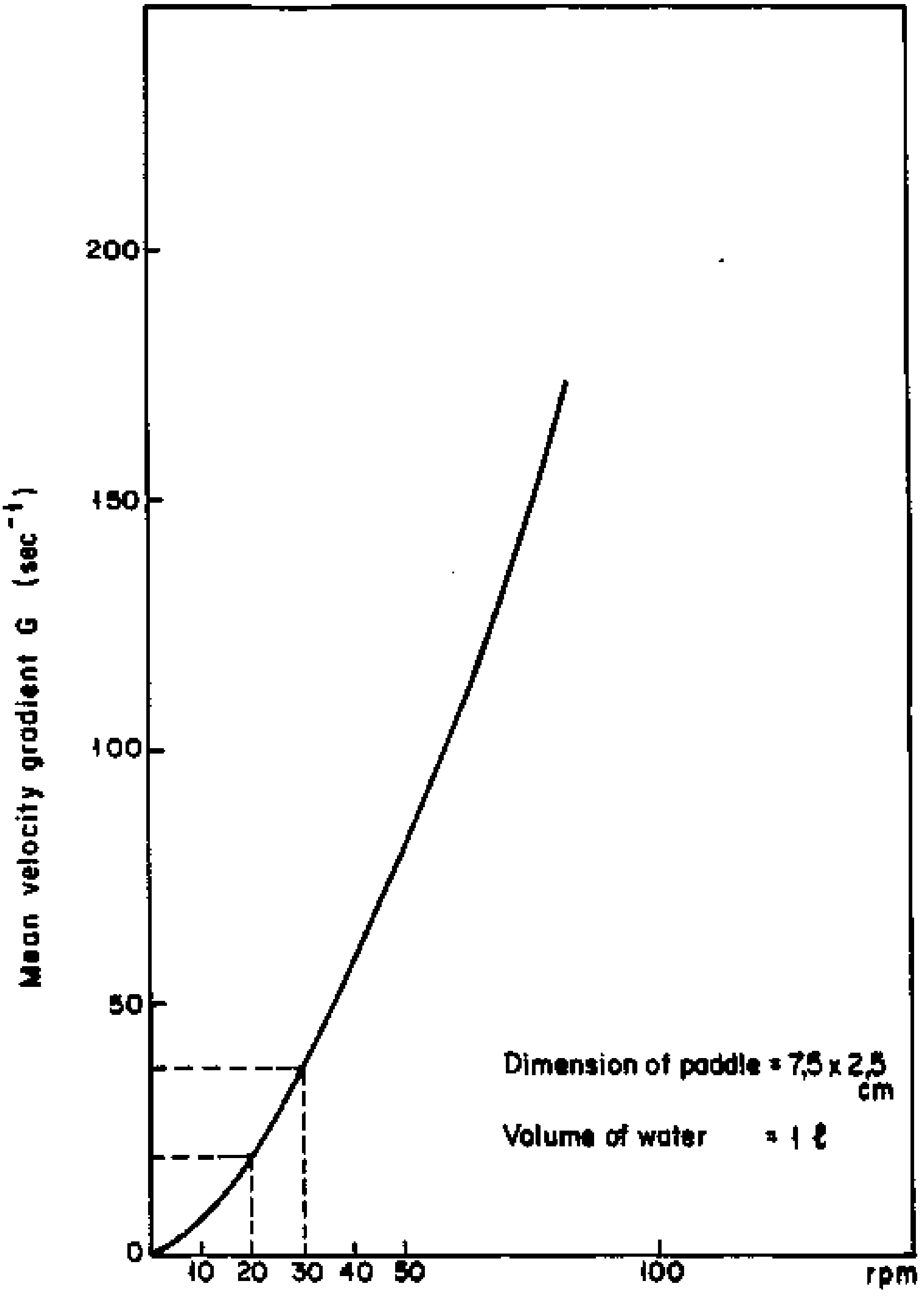


Figure 5.4: Relationship between paddle speed (rpm) and velocity gradient.

TABLE 5.2: Residual orthophosphate content (%) as a function of stirrer speed and $FeCl_3$ dosage (Screened raw sewage).

Dosage (mg/ℓ $FeCl_3$) / Stirrer speed (rpm)	10	20	30	35	40	45	50
75	43	31	34	47	45	42	40
85	38	27	25	44	40	38	40
90	33	25	24	42	36	38	37
100	30	22	18	34	31	18	32
120	22	18	10	21	18	18	17
130	21	16	9	18	13	15	140

5.3.3 Hydrolysis of condensed phosphates

Domestic sewage, in particular, contains a considerable amount of condensed phosphates which can be hydrolized during wastewater treatment (Finstein and Hunter, 1967; Heinke and Norman, 1969). The concentration of condensed phosphate in Daspoort settled sewage and the amount of orthophosphate which can derive from its hydrolysis are listed in Table 5.3.

Hydrolysis by strong acid treatment under pressure (138 kPa) caused an increase of orthophosphate in all cases. In screened raw sewage the amount of orthophosphate rose by 66 per cent. In the samples treated with $FeCl_3$ the amount of orthophosphate increased by 10 to 20 per cent after hydrolysis. Addition of $FeCl_3$ subsequent to initial dosage and hydrolysis resulted in removals from 96 to 99 per cent. To investigate whether this very good orthophosphate removal was due to the vigorous hydrolysis conditions, the responses of screened raw sewage and biological filter effluent to $FeCl_3$ treatment were compared. The results are listed in Table 5.4.

By dosing 120 mg/ℓ $FeCl_3$ to screened raw sewage an orthophosphate removal of 83 per cent was achieved which yielded 1,0 mg/ℓ (as P) in the effluent. Total phosphate could be removed down to 1,99 mg/ℓ (as P), which is equal to a removal of 82 per cent.

TABLE 5.3: Effect of $FeCl_3$ addition on phosphate concentration in Daspoort screened sewage, before and after hydrolysis of condensed phosphates.

Parameter Dosage (mg/ℓ $FeCl_3$)	Ortho-phosphate % residue	Condensed phosphate % residue (unhydrolysed)	Total phosphate % residue	pH
0	100	100	100	7,36
70	34	29	85	6,36
80	23	15	25	6,84
90	20	22	21	6,76
100	16	22	19	6,70
120	12	18	19	6,57
130	9	21	20	6,56

Hydrolysed supernatant

Initial dosage				
0	166			
70	46			
80	34			
90	34			
100	30			
120	23			
130	30			

Chemically treated hydrolysed supernatant

Chemical addition followed by hydrolysis and further chemical addition

Initial.	Additional				
(1)	70	4	0	23	6,88
(2)	80	2	0	9	6,60
(3)	90	2	0	6	6,33
(4)	100	1	0	8	5,99
(5)	120	1	0	11	5,08
(6)	130	1	0	12	4,56

The untreated biological filter effluent still contained 6,0 mg/ℓ orthophosphate (as P), which is 97 per cent of the concentration in the incoming sewage. The amount of total phosphate concentration was was 7,2 mg/ℓ (as P), equal to 67 per cent of the influent value. Biological filter effluent with these characteristics, upon the addition of 90 mg/ℓ $FeCl_3$ had its orthophosphate content reduced to 1,1 mg/ℓ (as P), which constitutes a removal of 82 per cent. Similarly, the total phosphate of 1,3 mg/ℓ (as P) in the effluent constituted an 82 % removal.

5.3.4 Removal of COD and TKN by $FeCl_3$ addition

In addition to phosphate, carbonaceous energy, here expressed as COD, can be removed by chemical addition. (Figure 5.5). The addition of ferric chloride at a Fe:P ratio of 1:1 was sufficient to remove about 50 per cent of the COD in the incoming raw sewage, but at a ratio of 2:1, 70 per cent of the COD was removed. The practical implications are that organic overloading of trickling filters may be reduced by chemical addition prior to the primary sedimentation tank, or that the hydraulic load can be increased, provided the minimum retention time required for the oxidation of carbonaceous and nitrogenous compounds within the biological filter is maintained.

Removal of nitrogenous compounds by $FeCl_3$ addition alone is insufficient to meet the standards required. Figure 5.4 shows that only about 30 per cent of the influent TKN could be removed by adding $FeCl_3$ at ratios up to 3,5:1 (Fe:P). This result, however, was already achieved at a Fe:P ratio of 1:1.

TABLE 5.4: Comparison of phosphate residuals after adding $FeCl_3$ to screened raw sewage and biological filter effluent.

$FeCl_3$ dosage (mg/ℓ)	Orthophosphate		Total phosphate	
	Screened raw sewage	Biological filter effluent	Screened raw sewage	Biological filter effluent
0	6,1	5,9	10,8	7,2
90	2,2	1,1	3,9	1,3
100	1,6	0,7	2,1	1,1
120	1,0	0,5	1,9	1,1

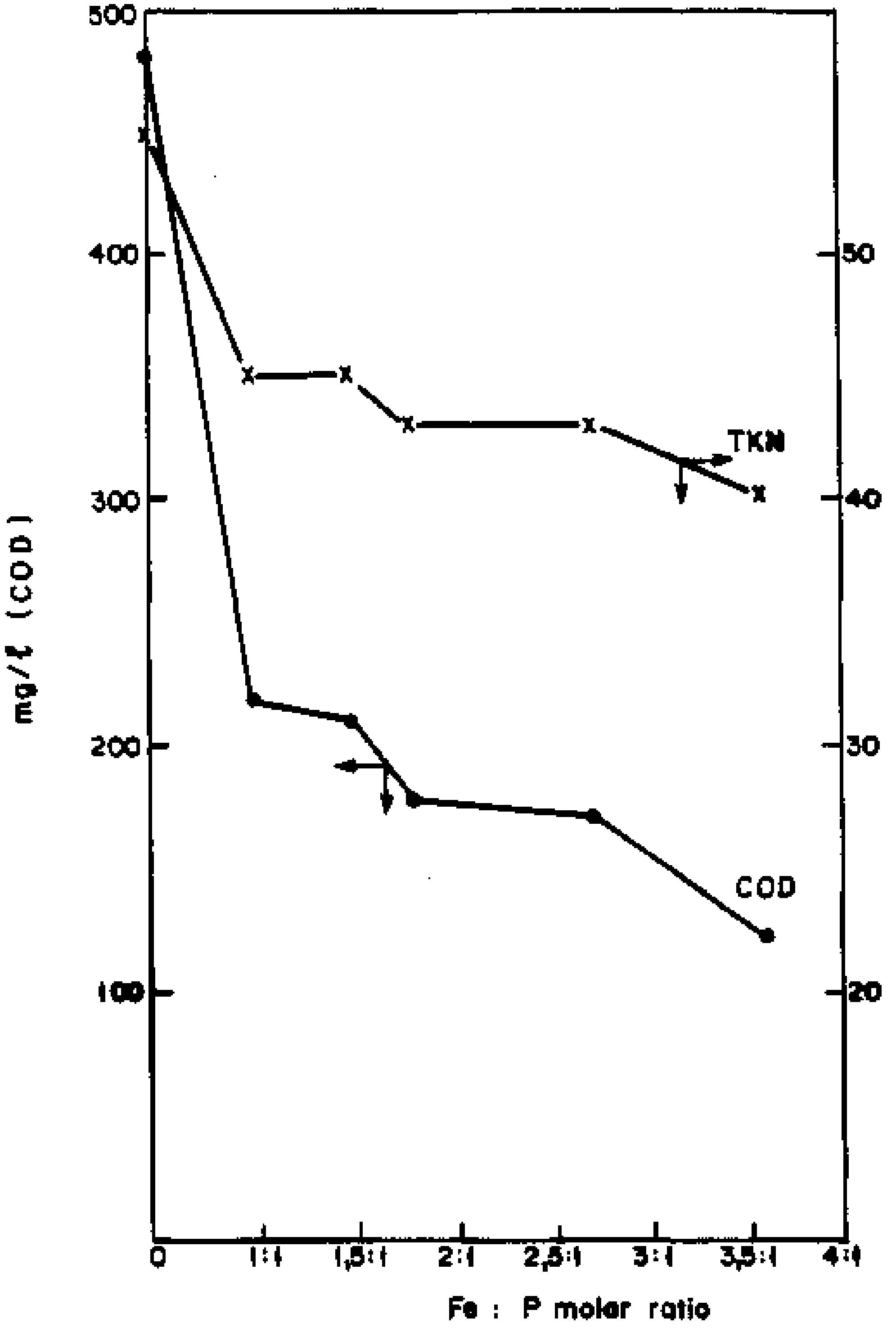


Figure 5.5. Removal of carbonaceous and nitrogenous compounds by adding ferric chloride to raw sewage.

demonstrates that treatment with iron salts does not bring down the total nitrogen concentration to the desired low level.

5.4 Discussion and conclusion.

Different degrees of success in phosphorus removal were achieved at different times during the day, coinciding with the varying concentrations of condensed phosphates in the influent sewage.

A remarkable amount of orthophosphate can be derived from condensed phosphate as a result of hydrolysis. This amount of orthophosphate must be taken into consideration, when the dosage of $FeCl_3$ required to remove sufficient phosphorus is calculated. Ketchum and Higgins (1979) reported that condensed phosphate had a much greater effect on increased metal dosage requirements than orthophosphate. This also explains why post-treatment by $FeCl_3$ was found to be so much more efficient than pre-treatment. Post-treatment should be chosen if phosphate removal alone is the objective. If, however, one is dealing with an organically overloaded system pre-treatment might be preferred. Up to 70 per cent of the COD in the incoming sewage was removed chemically by adding $FeCl_3$ in a Fe:P ratio of 2:1. This would either ameliorate the organic overload or allow the hydraulic load to be increased to the plants maximum capacity. Split-treatment may be the treatment of choice if both objectives should be met.

Nett nitrogen removal amounted to 30 per cent of the influent TKN. This emphasizes the need to consider other means, such as biological methods to meet the nitrogen standards.

Stirrer speed had a significant influence on the effectiveness of phosphorus removal by $FeCl_3$ precipitation. The reason for this is that, if metal salts are added to a phosphorus solution while stirring very effective phosphate adsorption takes place (Lijklema, 1979). This enables the metal ions to react with the phosphate before relatively unreactive OH-bonds are formed. On the other hand the stirrer speed should not be too high in order to prevent shearing of already formed flocs.

6. MASS BALANCE STUDIES FOR CARBON SOURCES AVAILABLE AT SEWAGE TREATMENT WORKS.

In contrast to the removal of phosphates and COD which can be achieved chemically, the removal of nitrogenous compounds is much more complex. Colloidal nitrogenous compounds may be removed by coagulation, while ammonia may be removed, for example, by line addition followed by air stipping or by precipitation as magnesium ammonium phosphate salts under carefully controlled conditions. Denitrification, however, is only feasible if done biologically. The combination of a biological filter system and an activated sludge process might be considered to upgrade existing biological filter systems in order to meet the nitrate standards. This process, however, requires carbonaceous energy in the form of COD. Chemical addition removes carbonaceous energy, which even in the absence of chemical treatment is a limiting substrate in most local wastewaters. The supplementation of energy has, therefore, to be considered. The use of external energy sources such as methanol or molasses may not in all cases be suitable. Research workers in Canada investigated the use of effluents from food processing industries, breweries and distilleries as potential substrates for biological denitrification (Monteith, Bridle and Sutton, 1979). However, no detailed information is available as yet. Therefore, the use of external carbon sources available at sewage treatment works is being considered.

As a preliminary study various potential energy sources available at the Daspoort sewage works were investigated regarding their suitability as energy supplements as far as C:N ratio and biodegradability are concerned. This research has then been expanded to mass balance studies at Rooiwal sewage treatment works, north of Pretoria and Vlakplaats sewage treatment works at Boksburg to evaluate the amounts in which these carbon sources are available.

6.1 Preliminary studies at Daspoort/Pretoria.

The following potential carbon sources were investigated. (For illustration, see Figure 6.1).

(a) Supernatant liquor from anaerobic digester No. 7. This digester which is mixed and heated, receives sludge from the primary settlers.

(b) Sludge from the bottom of digester No. 5. This is a primary digester receiving sludge from the units treating sewage of mostly domestic origin. This digester is also mixed and heated.

(c) Sludge from the bottom of digester No. 6. This is a secondary digester receiving sludge from digester No. 5.

(d) Supernatent liquor from digester No. 6.

All three digesters are two-stage 27,75 m diameter Dorr digesters with a total capacity of $906/m^3$. The primary digester is heated from 35 to 37,5 °C by direct injection of steam. The digester content is mixed by 6 pumps circulating sludge gas. The secondary digester is an open tank. Supernatant is drawn off only when necessary and if so until it becomes turbid (Thirion, personal communication).

(e) Raw sludge from biological filter units 1 to 6, which receive sewage of mostly domestic origin.

(f) Supernatant from 'rectangular tanks'. These tanks serve as secondary clarifiers for the digester effluents.

(g) Sludge from the bottom of the rectangular tanks.

(h) Mixed liquor from activated sludge units 9 to 11. These units treat sewage which also contains industrial effluents.

(i) Return sludge from activated sludge units 9 to 11.

(j) Humus from secondary humus tanks of the biological filter units 5 and 6.

(k) Centrifuge centrate. This centrifuge serves for dewatering of the sludges from digesters 1 to 5 and 7.

(l) Drying bed effluents.

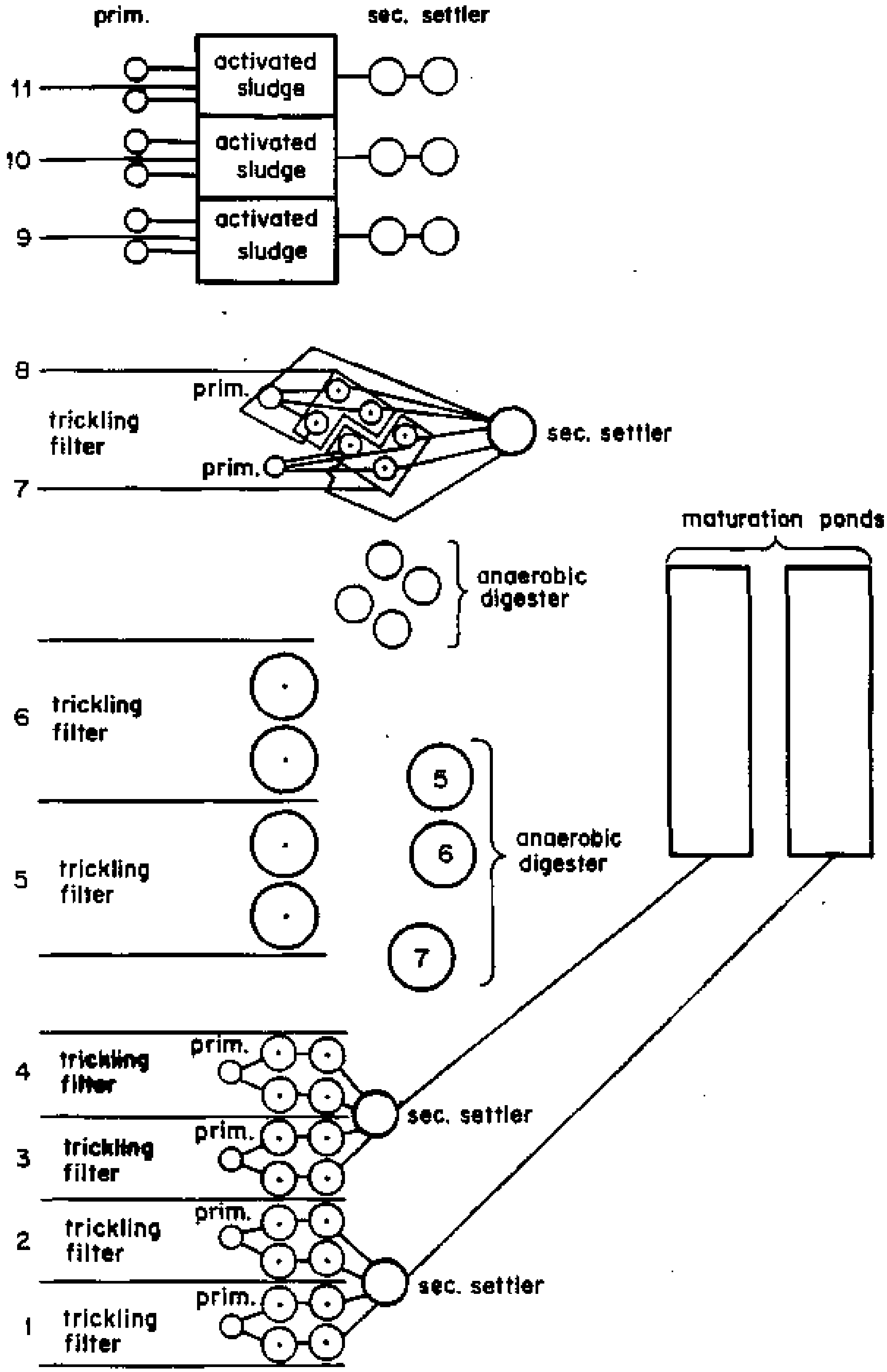


Figure 6.1: Layout of the Daspoort sewage purification works: Schematic of the various units.

Two sets of samples were collected on different weekdays at Daspoort sewage works. All the samples were analyzed for COD, TKN, total P and in the case of the second set, also for NH_4^+.

The first set of samples were analysed without prior filtration but those of the second set were analysed separately for the liquid and solid phases. For this purpose the samples were filtered through Whatman 2V filter paper. Mixed liquor suspended solids (MLSS) and VSS were also determined. The results of the chemical analyses and the C:N:P ratios are listed in Table 6.1 and 6.2, respectively.

TABLE 6.1: Chemical analyses of potential supplementary carbon sources for biological denitrification (Samples taken at Daspoort sewage treatment works).

Carbon Source	COD	TKN	Total P
BATCH 1 (1980-05-27)(all values are mg/ℓ)			
Supernatant liquor digester No. 7	2300	900	44
Primary digested sludge (digester No. 5)	33200	1900	370
Secondary digested sludge (digester No. 6)	57400	1800	400
Supernatant liquor digester No. 6	1500	500	46
Raw sludge units 1 to 6	10400	2500	130
Supernatant rectangular tanks	1800	900	52
Bottom sludge rectangular tanks	24200	3300	945
Mixed liquor units 9 to 11	2000	300	31
Return sludge units 9 to 11	2000	200	480
Humus from secondary humus tank units 5 and 6	234400	2900	580

	COD	TKN	NH_4^+	Total P	SS g/ℓ	VSS g/ℓ
	mg/kg sludge					
BATCH 2 (1980-06-12)						
(a) Total Sample						
Supernatant liquor digester No. 7	11600	1100	800	200	10,90	7,56
Primary digested sludge (digester No. 5)	33890	1500	700	350	43,25	29,54
Secondary digested sludge (digester No. 6)	40300	1800	500	450	44,38	30,09
Supernatant liquor (digester No. 6)	26300	400	400	350	0,51	0,77
Raw sludge units 1 to 6	50800	1900	1200	450	52,88	43,03
Supernatant rectangular tanks	735	700	500	350	0,78	0,62
Bottom sludge rectangular tanks	75300	2800	700	850	84,10	52,20
Mixed liquor units 9 to 11	2500	200	20	550	2,27	2,24

TABLE 6.1: (continued)

	COD	TKN	NH_4^+	Total P	SS g/ℓ	VSS g/ℓ
	mg/kg sludge					
Return sludge units 9 to 11	4200	300	20	675	3,88	3,06
Humus from secondary humus tank units 5 and 6	39900	2300	1300	550	35,27	25,01
Centrifuge centrate	6300	500	400	550	4,57	4,00
Drying bed effluent	30	400	400	150	0,06	0

(b) Soluble fraction (all values are mg/ℓ)

Carbon Source	COD	TKN	NH_4^+	Total P
Supernatant liquor (digester No. 7)	1100	1100	1000	20
Primary digested sludge (digester No. 5)	500	500	350	24
Secondary digested sludge (digester No. 6)	600	600	400	17
Supernatant liquor (digester No. 6)	300	400	400	30
Raw sludge units 1 to 6	7700	400	200	140
Supernatant rectangular tanks	300	700	600	16
Bottom sludge rectangular tanks	800	800	500	11
Mixed liquor units 9 to 11	40	20	4	39
Return sludge units 9 to 11	20	100	30	46
Humus from secondary humus tank units 5 and 6	1300	200	140	66
Centrifuge centrate	300	400	400	24
Drying bed effluent	200	400	370	16

(c) Solid fraction (all values are mg/kg dried sludge)

Carbon Source	COD	TKN	Total P
Supernatant liquor digester No. 7	274000	17000	12000
Primary digested sludge (digester No. 5)	34000	2450	750
Secondary digested sludge (digester No. 6)	18300	3300	800
Raw sludge units 1 to 6	36800	1300	700
Supernatant rectangular tanks	11100	47800	60600
Bottom sludge rectangular tanks	10650	2100	100
Mixed liquor units 9 to 11	725100	67300	28300
Return sludge units 9 to 11	3093500	65800	20600
Humus from secondary humus tank units 5 and 6	14000	5000	100
Centrifuge centrate	1297000	45200	31400

TABLE 6.2: C:N:P ratios of different potential supplementary carbon sources for biological denitrification sampled at Daspoort.

Carbon Source	C	N	P
BATCH 1			
Supernatant liquor digester No. 7	2,4	1	0,05
Primary digested sludge (digester No. 5)	17,5	1	0,20
Secondary digested sludge (digester No. 6)	31,2	1	0,20
Supernatant liquor digester No. 6	3,2	1	0,10
Raw sludge units 1 to 6	4,1	1	0,05
Supernatant rectangular tanks	1,9	1	0,05
Bottom sludge rectangular tanks	7,3	1	0,30
Mixed liquor units 9 to 11	7,5	1	0,12
Return sludge units 9 to 11	8,1	1	1,95
Humus from secondary humus tank units 5 and 6	80,0	1	0,20

TABLE 6.2: C:N:P ratios of different potential supplementary carbon sources for biological denitrification sampled at Daspoort.

Carbon Source	C	N	P	NH_4^+ (% of TKN)
BATCH 2				
(a) Total sample				
Supernatant liquor digester No. 7	10,8	1	0,18	74
Primary digested sludge (digester No. 5)	22,9	1	0,24	48
Secondary digested sludge (digester No. 6)	22,8	1	0,25	28
Supernatant liquor digester No. 6	85,9	1	0,74	100
Raw sludge units 1 to 6	26,5	1	0,23	64
Supernatant rectangular tanks	1,0	1	0,50	72
Bottom sludge rectangular tanks	26,7	1	0,30	24
Mixed liquor units 9 to 11	13,0	1	2,86	9
Return sludge units 9 to 11	15,5	1	0,40	6
Humus from secondary humus tank units 5 and 6	17,7	1	0,24	57
Centrifuge centrate	12,6	1	1,10	71
Drying bed effluent	0,1	1	0,40	95
(b) Soluble fraction				
Supernatant liquor digester No. 7	1	1	0,02	93
Primary digested sludge (digester No. 5)	1	1	0,05	73
Secondary digested sludge (digester No. 6)	1	1	0,05	65
Supernatant liquor digester No. 6	0,6	1	0,07	91
Raw sludge units 1 to 6	20,4	1	0,37	52
Supernatant rectangular tanks	0,4	1	0,02	93
Bottom sludge rectangular tanks	1	1	0,01	68
Mixed liquor units 9 to 11	1,7	1	1,73	16
Return sludge units 9 to 11	0,2	1	0,42	29
Humus from secondary humus tank units 5 and 6	6,9	1	0,34	72
Centrifuge centrate	0,7	1	0,06	94
Drying bed effluent 0,6	0,6	1	0,04	93

Carbon Source	C	N	P
(c) Solid fraction			
Supernatant liquor digester No. 7	16,1	1	0,74
Primary digested sludge (digester No. 5)	13,9	1	0,30
Secondary digested sludge (digester No. 6)	5,5	1	0,24
Supernatant liquor digester No. 6	-	-	-
Raw sludge units 1 to 6	27,3	1	0,52
Supernatant rectangular tanks	0,23	1	1,27
Bottom sludge rectangular tanks	5,1	1	0,05
Mixed liquor units 9 to 11	10,8	1	0,42
Return sludge units 9 to 11	46,9	1	0,31
Humus from secondary humus tank units 5 and 6	2,8	1	0,02
Centrifuge centrate	28,7	1	0,69
Drying bed effluent	-	-	-

The results of the two sets of samples exhibit wide differences among the various parameters. With respect to the C:N ratio bottom sludge from digesters No. 5 and 6 as well as humus from the secondary humus tanks of units 5 and 6 were the carbon sources in the first set of samples which were considered potentially suitable as supplementary energy sources. In the second set all the samples showed a C:N ratio higher than 10:1. All the soluble fractions showed a C:N ratio lower than 10:1 except for raw sludge from units 1 to 6.

The ammonia concentration was very high in all the samples. It constituted upwards of 75 % of the TKN. There were only three exceptions: raw sludge from units 1 to 6 with ammonia constituting 52 % of the TKN in the soluble fraction, mixed liquor from units 9 to 11 with 16 % and return sludge from units 9 to 11 with 22 % . Another factor of extreme importance relates to the biodegradability of these energy sources. For these investigations only the most promising carbon sources, with respect to C:N ratio and availability, were used: raw sludge, fed to a primary digester (digester No. 5), primary digested sludge, going from digester No. 5 to the secondary digester No. 6, bottom sludge and supernatant from digester No. 6. All these samples were monitored in unfiltered form for COD, TKN, NH_4^+ and total P. The liquid and solid fractions were separated again, this time, however, by centrifuging for 20 min at 5000 rpm. Two separate sets of samples were analyzed, which again showed large differences in composition as summarized in Table 6.3.

Biodegradability of the different carbon sources was tested using the Warburg technique (Umbreit, Burns and Stauffer, 1959), by measuring the oxygen consumption over a defined time period under constant conditions. Glucose in a concentration of 2 % was monitored as a reference. The temperature during the experiment was kept constant at 25 °C and the reaction vessels were shaken at a frequency of 50 rpm. From each carbon source a sample of 3 mℓ was taken and the experiment started by inoculation with 1 mℓ mixed liquor from the aerated reactor of an activated sludge laboratoryscale unit. As a control distilled water inoculated with mixed liquor was used. Supernatant from digester No. 6 showed good biodegradability in the first set, compared to the results obtained with glucose. The second set showed poor biodegradability. There was also a difference between these two sets with regard to C:N:P ratio (Table 6.4). The first batch showed a ratio which was favourable for biological denitrification, but the same good result was not obtained for the second set. The overflow from digester No. 5 to digester No. 6 also showed poor biodegradability in both sets of samples. The C:N:P ratio was also favourable in both cases.

TABLE 6.3: Chemical analyses of potential external carbon sources at Daspoort sewage treatment works (Sample separation by centrifugation)

Carbon Source	COD g/ℓ	TKN mg/ℓ	NH_4^+ mg/ℓ	Total P mg/ℓ	SS g/ℓ	VSS g/ℓ
BATCH 1						
(a) Total sample						
Raw sludge	70	360	160	2300	60	50
Primary digested sludge	70	310	200	2400	25	50
Secondary digested sludge	50	310	290	2300	60	40
Supernatant secondary digester	1	220	70	2300	1	1
(b) Supernatant after centrifugation						
Raw sludge	7	340	240	140	-	-
Primary digested sludge	3	460	370	60	-	-
Secondary digested sludge	3	480	450	60	-	-
Secondary digester sludge	3	410	350	80	-	-
(c) Pellet after centrifugation (all values g/kg dried sludge)						
Raw sludge	420	40	-	10	-	-
Primary digested sludge	350	20	-	10	-	-
Secondary digested sludge	400	30	-	10	-	-
Secondary digester supernatant	500	40	-	10	-	-

TABLE 6.3 (continued)

Carbon source	COD g/ℓ	TKN mg/ℓ	NH_4^+ mg/ℓ	Total P mg/ℓ	SS g/ℓ	VSS g/ℓ
BATCH 1						
(a) Total sample						
Raw sludge	50	200	200	300	30	25
Primary digested sludge	20	220	160	300	20	20
Secondary digested sludge	30	420	200	400	40	30
Supernatant secondary digester	5	550	110	80	2	1
(b) Supernatant after centrifugation						
Raw sludge	2	170	150	80	-	-
Primary digested sludge	1	300	300	20	-	-
Secondary digested sludge	1	200	200	20	-	-
Secondary digester supernatant	1	200	160	30	-	-
(c) Pellet after centrifugation (all values g/kg dried sludge)						
Raw sludge	900	100	-	10	-	-
Primary digested sludge	600	80	-	10	-	-
Secondary digested sludge	700	60	-	10	-	-
Secondary digester supernatant	200	130	-	10	-	-

TABLE 6.4: C:N:P ratios of different carbon sources investigated

	C	N	P	% NH_4^+ (of TKN)
BATCH 1				
(a) Total sample				
Raw sludge	194	1	6,3	43
Primary digested sludge	223	1	7,9	66
Secondary digested sludge	157	1	7,4	94
Secondary digester supernatant	4,5	1	10,4	30
(b) Supernatant after centrifugation				
Raw sludge	20,5	1	0,4	70
Primary digested sludge	6,5	1	0,1	80
Secondary digested sludge	5,4	1	0,1	94
Secondary digester supernatant	6,3	1	0,2	85
(c) Pellet after centrifugation				
Raw sludge	9,8	1	0,2	-
Primary digested sludge	15,3	1	0,4	-
Secondary digested sludge	12,2	1	0,3	-
Secondary digester supernatant	13,5	1	0,3	-

TABLE 6.4

	C	N	P	% NH_4^+ (of TKN)
BATCH 2				
(a) Total sample				
Raw sludge	240	1	1,6	100
Primary digested sludge	71,4	1	1,2	71
Secondary digested sludge	66,7	1	0,9	48
Secondary digester supernatant	7,6	1	0,1	17
(b) Supernatant after centrifugation				
Raw sludge	10,3	1	0,5	92
Primary digested sludge	0,9	1	0,07	96
Secondary digested sludge	1,3	1	0,1	97
Secondary digester supernatant	1,02	1	0,1	82
(c) Pellet after centrifugation				
Raw sludge	7,3	1	0,1	-
Primary digested sludge	7,6	1	0,1	-
Secondary digested sludge	12,0	1	0,1	-
Secondary digester supernatant	13,0	1	0,1	-

6.2 Mass balance for potential carbon sources at Rooiwal sewage treatment works.

The carbon sources which seemed to be the most promising in the Daspoort preliminary studies were chosen to be evaluated in a mass balance study conducted at Rooiwal sewage treatment works (Figure 6.2). The purpose of this study was to determine the composition and quality of locally available raw sludge, primary and secondary digested sludge as well as digester supernatant. The Rooiwal sewage treatment works - the east works - north of Pretoria is operated under fairly constant conditions in contrast to the Daspoort sewage works. The east works treat a daily average flow of 54 Mℓ sewage which is of mainly domestic origin. The industrial portion, which constitutes

1 Mℓ/day, is composed of 0,9 % tannery effluent, 0,6 % of metal engineering works effluent and 0,4 % pharmaceutical industry effluent (percentage of total flow). The raw sludge production rate at Rooiwal amounts to 1 % of the incoming flow.

Samples were taken daily over a four week period. Raw sewage was sampled hourly and composited in proportion to flow over 24 h. Settled sewage was sampled two-hourly and otherwise treated similarly. For raw, primary and secondary digested sludge one sample was taken from every sump filled and composited.

Table 6.5 shows the average concentrations of parameters of interest for raw and settled sewage, raw, primary digested and secondary digested sludge. Table 6.6 shows the C:N:P ratios, the ratio of NH_4^+ to TKN and the ratio of orthophosphate to total phosphate. It is obvious that the C:N ratios of raw sludge and the two digested sludges were very favourable as potential energy sources for biological denitrification. As far as biodegradability was concerned raw sludge showed the best results, followed by primary digested and secondary digested sludge. During the period of sampling, no digester supernatant was generated.

To calculate the energy requirements for biological denitrification the equation of MacCarty, Beck and St. Amant (1969) was used, based on methanol as an energy source:

$$C_M = 2{,}47 \cdot N_o + 1{,}53 \cdot N_1 + 0{,}87 \cdot D_o$$

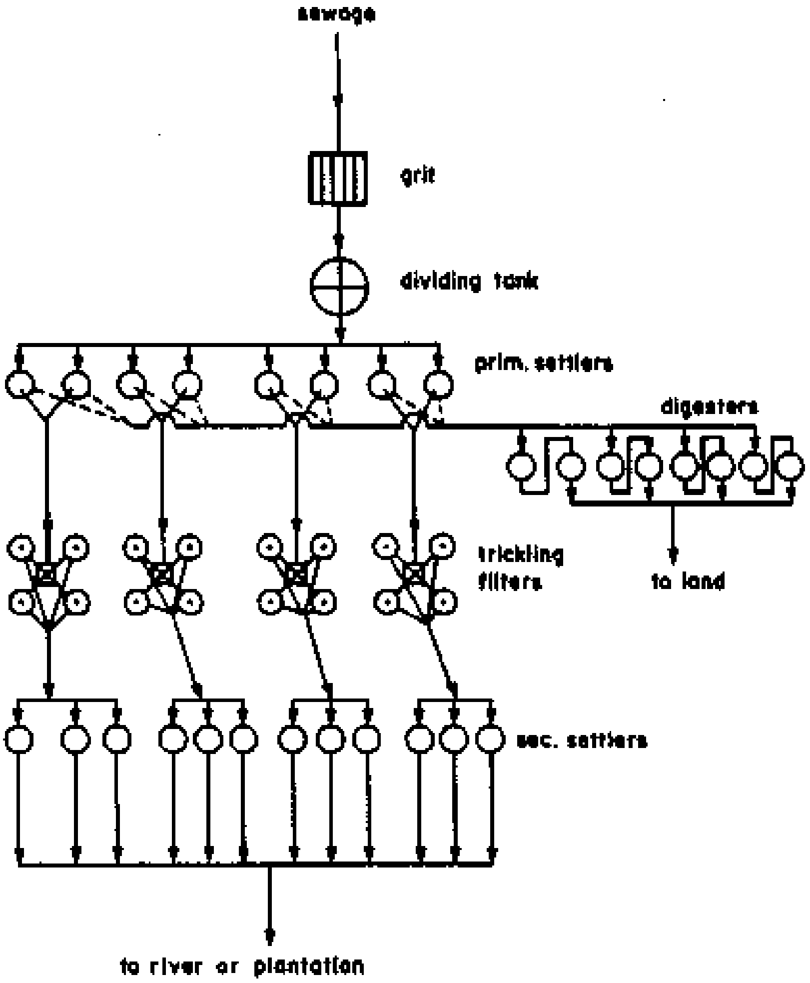
where C_M = methanol concentration, mg/ℓ

N_o = nitrate concentration, mg/ℓ

N_1 = nitrite concentration, mg/ℓ

DO = dissolved oxygen, mg/ℓ .

For biological filter effluents with 15 mg/ℓ nitrate (as N) 0,5 mg/ℓ nitrite (as N) and an assumed DO of 2,0 mg/ℓ the required methanol concentration would be 39,5 mg/ℓ for complete denitrification. The COD equivalent of 1 mg/ℓ methanol is 1,5 mg/ℓ COD, which gives a COD requirement of 60 mg/ℓ for complete denitrification, or a 4:1 COD : NO_3-N ratio.



prim. settlers : 13m diameter Dortmund tanks; 13m depth; 850 m^3 ; 3 h retention time.

filter : 4 x 4 clover leaf ; 36m diameter ; 3,7m depth ; 3875 m^3 each ; 37mm x 75mm slag media.

sec. settlers : 12m diameter Dortmund tanks ; 4 h retention time.

digester: 2124 m^3, high-rate type ; 16m diameter, 17m depth; 34° conical bottom; coned top; heated by live steam injection to 32°C – 36°C ; drought tube mixing.

Figure 6.2. Layout of Rooiwal East Works

TABLE 6.5: Parameter values for various carbon sources potentially suitable as energy sources for biological denitrification (Rooiwal Sewage Treatment Works)

Variable	Mean value mg/ℓ	Minimum mg/ℓ	Maximum mg/ℓ
Raw sewage			
COD	580	130	800
TKN	51	34	76
NH_4^+ (unfiltered)	33	14	44
NH_4^+ (filtered)	21	12	36
NO_3^--NO_2^-	0,5	0,2	1
Total N (unfiltered)	51	35	76
PO_4^{3-}	8	6	12
Total P	11	6	14
Settled sewage			
COD	476	180	890
TKN	48	36	63
NH_4^+ (unfiltered)	35	23	44
NH_4^+ (filtered)	26	13	46
NO_3^--NO_2^-	0,4	0,2	1
Total N (unfiltered)	48	36	63
PO_4^{3-}	8	5	11
Total P	11	8	17
Raw sludge			
COD	44400	10350	62550
TKN	1700	850	2480
NH_4^+ (filtered)	180	95	260
NO_3^--NO_2^-	0	0	0
PO_4^{3-}	160	95	210
Primary digested sludge			
COD	30100	14800	47350
TKN	1900	1580	2280
NH_4^+ (filtered)	530	48	640
NO_3^--NO_2^-	0	0	0
PO_4^{3-}	140	105	16
Total P	540	320	660

TABLE 6.5: (continued)

Variable	Mean value mg/ℓ	Minimum mg/ℓ	Maximum mg/ℓ
Secondary digested sludge			
COD	24250	13450	34500
TKN	1710	1160	2280
NH_4^+ (filtered)	570	350	700
NO_3^--NO_2^-	0	0	0
PO_4^{3-}	100	80	140
Total P	430	260	540

TABLE 6.6: C:N, N:P, ammonia and orthophosphate portion of potential carbon sources at Rooiwal Sewage Treatment Works.

Carbon source	C:N	P:N	NH_4^+ (unfilt.):TKN	PO_4^{3-}:TP
Raw sewage	11:1	0.2:1	0.6:1	0.8:1
Settled sewage	10:1	0.2:1	0.7:1	0.8:1
Raw sludge	25:1	0.26:1	-	0.3:1
Primary digested sludge	16:1	0.3:1	-	0.24:1
Secondary digested sludge	14:1	0.24:1	-	0.23:1

6.2.1 Raw sludge

During the sampling period 0,5 Mℓ/day of raw sludge had been produced. The COD was 44 000 mg/ℓ and TKN was 1 700 mg/ℓ (as N). If raw sludge is used as carbon source for denitrification one must be aware that the oxidation of TKN yields nitrates which should be removed as well. Assuming a 10:1 C:N requirement 17 000 mg/ℓ COD will be consumed for this purpose. The unbiodegradable fraction of the COD in raw sludge is assumed to be 15 % or 6 600 mg/ℓ .

Therefore, the COD remaining for denitrification will be:

$$44\ 000 - 17\ 000 - 6\ 600 = 20\ 400 \text{ mg/ℓ} .$$

As 60 mg/ℓ COD is required for complete denitrification of the remaining nitrate-nitrite, $2{,}94.10^{-3}$ ℓ raw sludge per ℓ biological filter effluent is necessary, which is equal to 0,14 Mℓ/day. This shows, that under the assumptions stated, ample raw sludge is available at Rooiwal for complete denitrification of biological filter effluent. This estimation, however, depends amongs others on the correctness of the estimate of the unbiodegradable fraction of the COD. If the unbiodegradable fraction is estimated higher, the following figures are obtained :

Unbiodegradable fraction	Raw sludge requirement
20 %	0,16 Mℓ/day
30 %	0,21 Mℓ/day
40 %	0,30 Mℓ/day
50 %	0,57 Mℓ/day

It is obvious that even with an unbiodegradable fraction as high as 40 %, which is considered most unlikely, the denitrification energy requirement can still be met.

6.2.2 Primary digested sludge

Based on the calculations mentioned above the following figures were obtained for primary digested sludge: a COD of 31 000 mg/ℓ and a TKN of 1 900 mg/ℓ (as N) were measured. The sludge production amounted to 0,37 Mℓ/day. If a 15 % unbiodegradable fraction is assumed the requirement for primary digested sludge would be 0,39 Mℓ/day, which approximates the volume which is daily available. In the case of primary digested sludge a higher unbiodegradable fraction must be assumed. For 20 % unbiodegradables 0,48 Mℓ/day would be required and in the case of 30 % unbiodegradables 0,95 Mℓ/day. Primary digested sludge is therefore considered unlikely to meet the energy requirements for complete denitrification of biological filter effluent.

6.2.3 Secondary digested sludge

The secondary digested sludge analysed at Rooiwal had a COD of 24 000 mg/ℓ and a TKN of 1 700 mg/ℓ . A total volume of 0,43 Mℓ/day is available. For 15 % unbiodegradables 0,81 Mℓ/day secondary digested sludge would be required and for 20 % 1,3 Mℓ/day. Again, the true unbiodegradable fraction is unknown. The results, however, show that secondary digested sludge is not suitable as an external carbon source for biological denitrification.

6.3 Mass balance at Vlakplaats Sewage Purification Works, Boksburg.

The results obtained from the Rooiwal mass balance were compared with those of a similar study performed at the Vlakplaats Sewage Purification Works at Boksburg. The lay-out of this works and the sampling points are given in Figure 6.3. The survey was conducted over 4 weeks and daily samples were composited according to flow. Mercury chloride was used for preservation.

Table 6.7 shows the mean values of parameters of interest at different sampling points and the quantities available of the different potential carbon sources. In Table 6.8 the C:N ratios are indicated. Similar to the findings from the Rooiwal study, raw and digested sludge seemed to be the most promising sources for external energy supplementation for biological denitrification. The mixture of humus sludge and drying bed supernatant, which is recycled to the head of the works (Figure 6.3), had a C:N ratio of 15:1. The samples taken from the different drying beds had been omitted from the calculation as for each drying bed only one or two analyses were available, which did not allow the calculation of statistically relevant mean values.

Assuming an unbiodegradable fraction of 15 % similar calculations to those given above show that the raw sludge requirement for complete biological denitrification of humus tank effluent would be 21,5 m^3/day. During the period of monitoring 113,04 m^3 raw sludge was produced daily. Digested sludge and the humus sludge-drying bed supernatant mixture were not available in sufficient quantities.

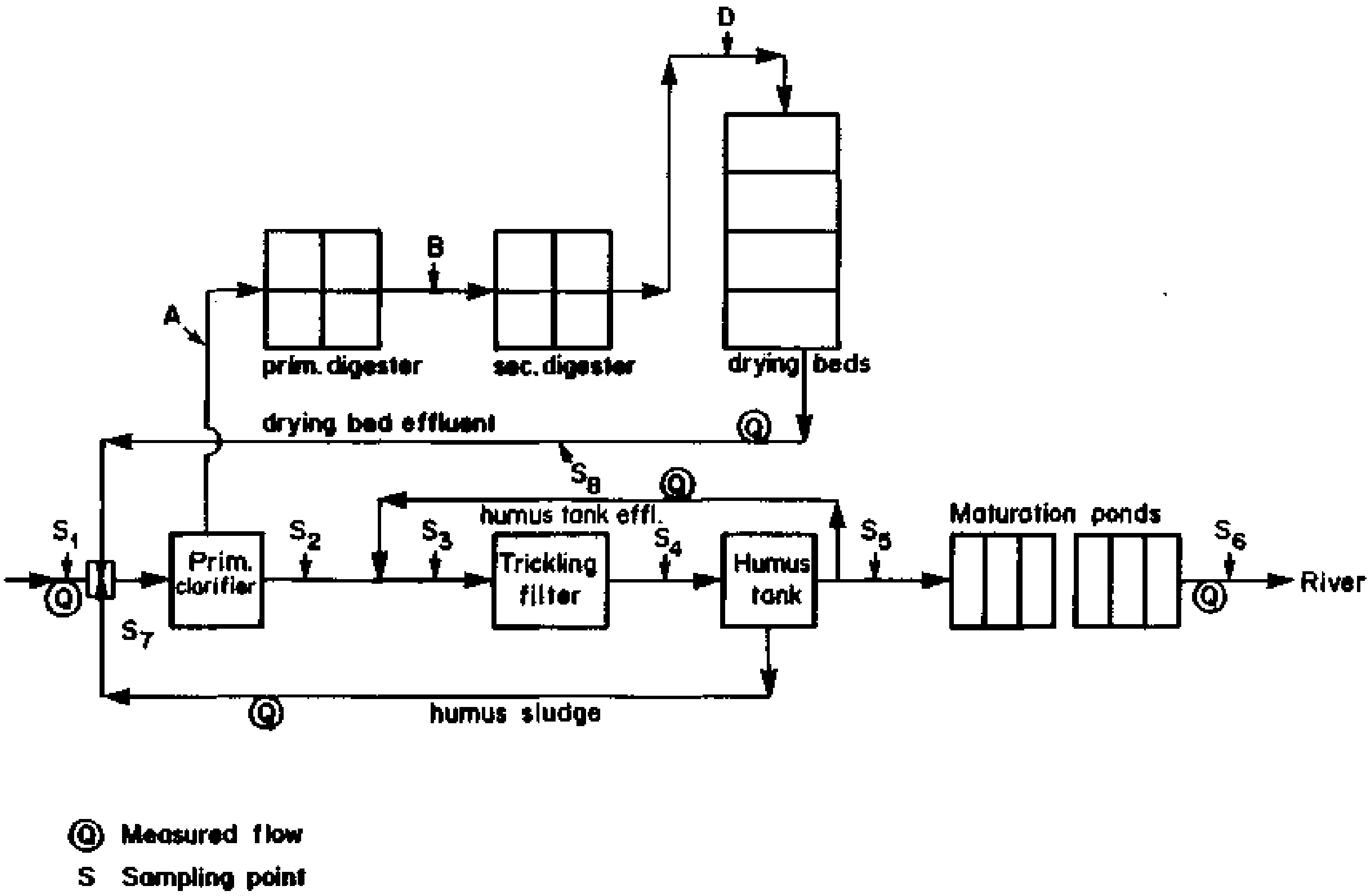


Figure 6.3: Lay-out of Vlakplaats Sewage Purification Works, Boksburg.

TABLE 6.7: Parameter values for carbon sources potentially suitable as energy sources for biological denitrification at Vlakplaats Sewage Purification Works (mean values in mg/ℓ)

Sampling point and number	Quantities available (m^3/day)	COD (unfilt.)	TKN (unfilt.)	NH_4^+ (filt.)	NO_3^- (filt.)	PO_4^{3-} (filt.)	TP (unfilt.)
Raw sewage (1)	21300	810	46	13	4	4	11
Settled sewage (2)		360	29	16	1	4	6
Settled sewage and recycled humus tank effluent (3)	15800	430	26	11	1	5	7
Biological filter effluent (4)		100	6	1	6	5	7
Humus tank effluent (5)		100	5	1	7	5	6
Final effluent after ponds (6)	39600	100	6	2	3	3	4
Humus sludge and drying bed supernatant (7)	1270	1100	76	39	2	6	27
Drying bed supernatant (18)	320	6350	940	475	0,4	15	39
Raw sludge (A)	110	84450	2800	264	0,3	91	565
Primary digester sludge (B)	110	44100	1170	682	0,2	25	496

TABLE 6.7: Parameter values for carbon sources potentially suitable as energy sources for biological denitrification at Vlakplaats Sewage Purification Works (mean values in mg/ℓ) (cont.)

Sampling point and number	Quantities available (m^3/day)	COD (unfilt.)	TKN (unfilt.)	NH_4^+ (filt.)	NO_3^- (filt.)	PO_4^{3-} (filt.)	TP (unfilt.)
Sec. dig. sludge to drying beds:							
Drying bed 1 (D1)	195	117600	3200	1000	0,2	64	490
Drying bed 2 (D2)	195	81200	3300	900	0,2	30	320
Drying bed 3 (D3)	195	36000	1900	500	0,4	23	440
Drying bed 4 (D4)	195	9600	140	100	0,3	12	80
Drying bed 7 (D7)	195	104100	5460	800	0,1	16	3000
Drying bed 8 (D8)	195	60400	2500	1000	0,2	40	1880
Drying bed 9 (D9)	195	197600	2650	1000	0,2	15	1440
Drying bed 10 (D10)	195	60700	2550	800	0,3	60	1160

For digested sludge 36,7 m^3/day is required but only 28,3 m^3/day is available, in the case of humus sludge plus drying bed supernatant 4 578 m^3/day is required and 1 270 m^3 is available.

TABLE 6.8: C:N ratios of potential external carton sources for biological denitrification at Vlakplaats Sewage Purification Works.

Parameter	COD:TKN
Raw sewage	18:1
Settled sewage	12:1
Settled sewage and recycled humus tank effluent	16:1
Humus sludge and drying bed supernatant	15:1
Drying bed supernatant	7:1
Raw sludge	30:1
Primary digested sludge	38:1

6.4 Discussion and Conclusion

The studies on the use of external carbon sources for energy supplementation of biological denitrification of biological filter effluent show that raw sludge is the most promising energy source. The quantities in which raw sludge is available as well as its C:N ratio is favourable. Although the use of digested sludge seemed feasible in the Rooiwal study this source was not available in sufficient quantities in the Vlakplaats study. Digester supernatant was not produced at Rooiwal during the period of monitoring. All the calculations on the required quantities of external energy sources were based on the assumption that complete denitrification of biological filter effluent must be achieved. However, this may not be necessary. Therefore, carbon sources like digested sludges might well be a proposition in cases where only partial denitrification is required. On the other hand digested sludges will contain a higher unbiodegradable fraction of COD than the 15 % assumed in the calculations, 40% may reflect a more realistic figure. This however, can only be determined in bench scale or pilot scale experiments.

7. PILOT PLANT STUDIES ON THE USE OF EXTERNAL CARBON SOURCES FOR BIOLOGICAL DENITRIFICATION

To investigate whether the findings obtained from the studies at Daspoort and the mass balances at Rooiwal and Vlakplaats, respectively, confirmed in practical experiments a pilot plant was operated using raw sludge, screened raw sewage and settled sewage respectively as external carbon sources for the denitrification of biological filter effluent.

7.1 Plant specifications

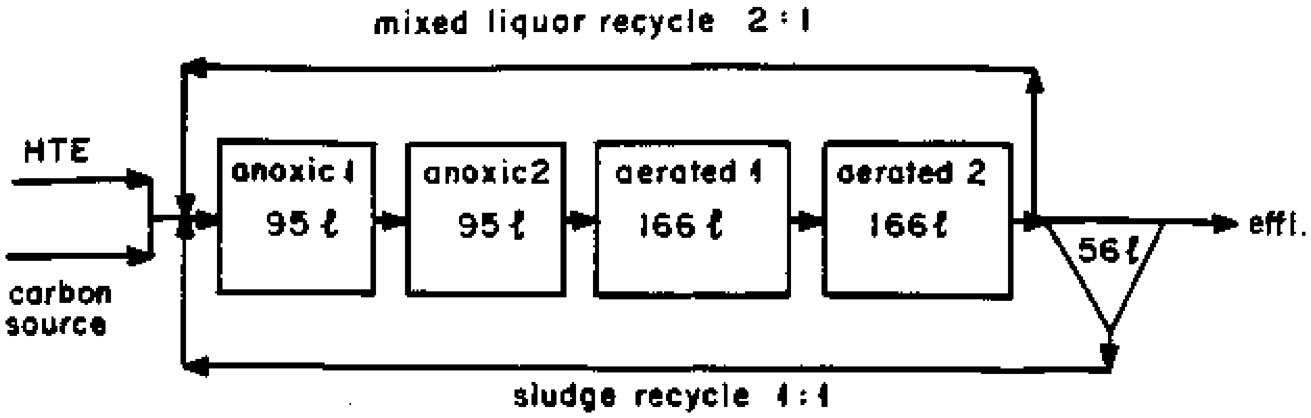
The specifications of this pilot plant are given in Table 7.1 and the lay-out is given in Figure 7.1.

TABLE 7.1: Pilot plant physical data during three different experimental periods in the study aimed at evaluating different carbon sources for biological denitrification.

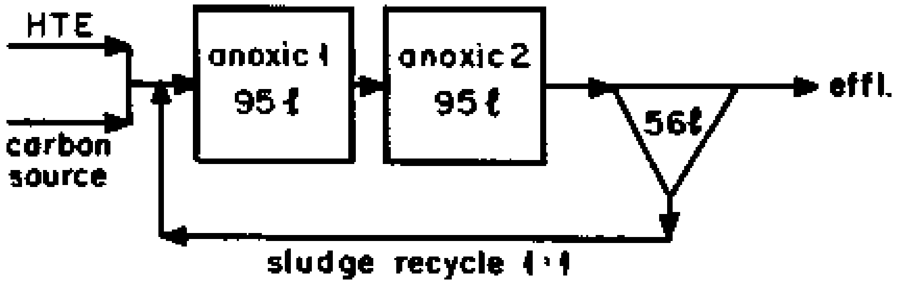
Parameter	Raw sludge feed	Raw sludge feed	Screened raw sewage and settled sewage feed
Total volume, ℓ	552	190	95
Aerated volume, ℓ	2 x 116	-	-
Anoxic volume, ℓ	2 x 95	2 x 95	95
Clarifier, ℓ	56	56	168
Clarifier surface area, m^2	0,15	0,15	0,38
Total feed flow, m^3/day	1,2	1,2	1,2
Sludge age, days	20	20	about 1

7.2 Activated sludge system with raw sludge as carbon source (1981-06-22 to 1981-10-31)

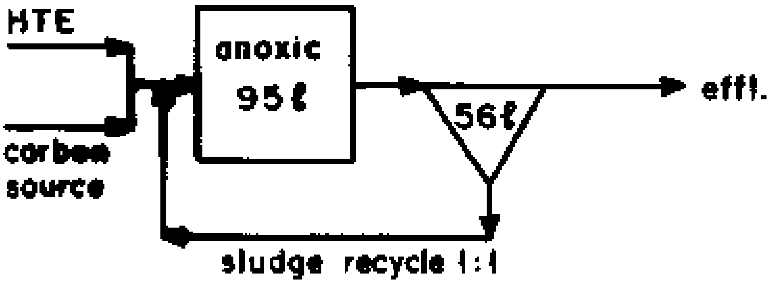
For the first experiment the pilot plant was operated as an activated sludge unit, comprising aerated and anoxic stages (Figure 7.1). The rational of this lay-out was that all nitrogenous components should be converted to nitrates. The feed comprised of two flows of equal magnitude, namely 0,6 m^3/day each. The first feed flow was humus tank effluent (HTE) which was drawn directly from the municipal biological filter and was fed continuously into a holding tank. The use of that



1. experiment : carbon source : raw sludge



2. experiment : carbon source : raw sludge



3. experiment : carbon source : screened raw sewage
4. experiment : carbon source : settled sewage

Figure 7.1: Lay-out of the pilot plant at different experiments.

tank had the same effect as a balancing tank in that concentration peaks were attenuated. The second feed flow was the carbon source, in this experiment raw sludge diluted with HTE. This feed was drawn from one of two holding tanks of approximately 1,4 m^3 capacity, and fed directly into the reactor. Each tank was used to supply carbon feed for two days, except over weekends when the current tank supplied feed for the period from Saturday to Monday.

The carbon feedstock was made up by diluting a volume of raw sludge, collected from a discharge sump in the municipal works, with HTE. From initial tests the humus tank effluent was found to have the following characteristics: a COD of 55 mg/ℓ, nitrite-nitrates amounted to 12 mg/ℓ (as N) and ammonia-nitrogen was 15 mg/ℓ (as N). From the Rooiwal analyses the quality of raw sludge was taken to have a COD of 44 000 mg/ℓ and a TKN of 1 700 mg/ℓ (as N). No nitrates were assumed to be present. In this phase of the experiment it was intended to use all raw sludge available per day for denitrification.

In order to provide a carbon source feed with a COD of approximately 1 000 mg/ℓ the raw sludge was diluted in the ratio 1:45.7 (raw sludge : HTE). This dilution step was necessary to make the raw sludge more readily pumpable. The theoretical composition after diluting the raw sludge with HTE in this proportion was then expected to be a COD of 996 mg/ℓ, a TKN of 51 mg/ℓ (as N) and a nitrate concentration of 11,7 mg/ℓ (as N). This carbon source was then pumped into the reactor at the same rate as the humus tank effluent feed and therefore the final theoretical quality of feed to the reactor was the average of these two flows.

The operation of the pilot plant started on 22 June 1981. The plant was seeded using mixed liquor taken from the Phoredox pilot plant and provided an initial mixed liquor suspended solids (MLSS) concentration of approximately 1 000 mg/ℓ . Sludge wasting was commenced on 9 July 1981 at a rate of 26 ℓ per day, thus providing a sludge age of 20 days.

The dissolved oxygen concentration in the aeration zone was maintained within the range of 2 to 3 mg/ℓ by means of a hand operated pressure regulator.

Table 7.2 shows the results of chemical analyses of parameters of interest in the different stages of the pilot plant. The quality of raw sludge varied considerably as far as COD was concerned. The values changed between 30 000 and 320 000 mg/ℓ and a mean value of 89 586 mg/ℓ was determined. The feedstock COD, i.e. the prediluted raw sludge, however, showed much less variation. Similarly, the feedstock TKN remained quite stable. The C:N ratio (COD:TKN), during this phase averaged 25:1.

TABLE 7.2: Average performance of an activated sludge unit receiving humus tank effluent when using raw sludge as a carbon and energy source (1981-06-22 - 1981-10-31) (concentrations in mg/ℓ)

	COD (unfilt)	TKN (unfilt.)	NH_4^+ (filtered)	$NO_2^- - NO_3^-$ (filtered)	PO_4^{3-} (filtered)
Raw sludge	89700	ND	ND	ND	ND
Diluted raw sludge (feedstock)	890	37,5	13,0	ND	17,9
Humus tank effluent	44	ND	9,7	11,9	9,4
Feed entering pilot plant	467	ND	11,4	5,9	13,6
Pilot plant effluent	42	ND	2,6	5,0	8,7

ND = Not determined.

The MLSS concentration increased rapidly a mean value of 4 611 mg/ℓ . This rapid increase may be ascribed to the highly favourable C:N ratio. During the whole experimental period the pilot plant performed very well as can be seen from Table 7.2 and Figure 7.2. The effluent COD averaged 42 mg/ℓ, while the ammonia was reduced to 2,6 mg/ℓ (as N). The nitrate-nitrite and orthophosphate concentrations in the effluent was 5,0 mg/ℓ (as N) and 8,7 mg/ℓ (as P), respectively.

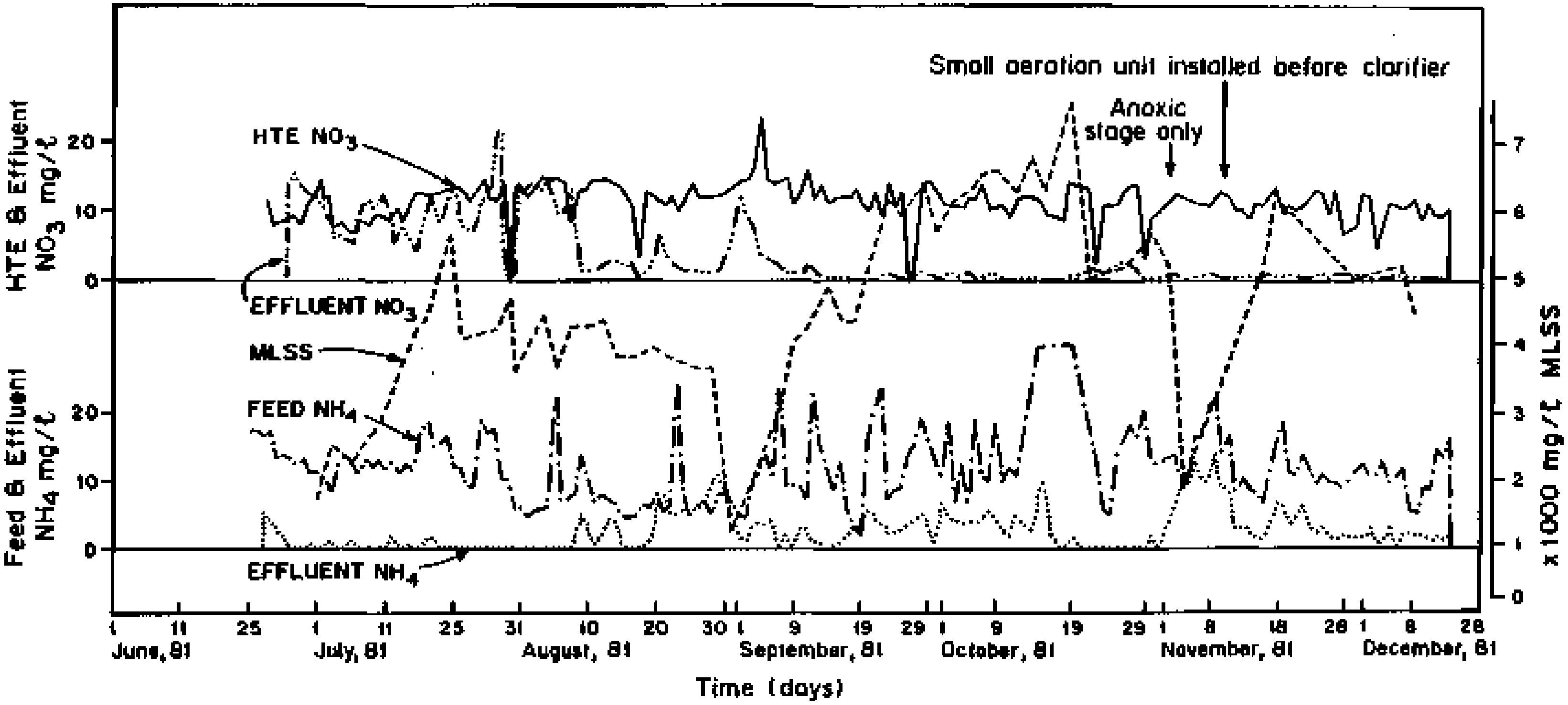


Figure 7.2: Pilot plant performance 1981-06-23 until 1981-12-14.

7.3 Denitrification unit with raw sludge as carbon source (1981-11-03 - 1981-12-14).

On 2 November 1981 the pilot plant was changed to a strict denitrification unit by by-passing the aeration tank. (Figure 7.1).

The change in the configuration resulted in a sharp drop in the MLSS concentration, (Table 7.3, Figure 7.3a), which was most probably due to adaptation and equilibration processes. The settling properties of the sludge in the clarifier deteriorated after omitting the aerated stage. This is expressed in the increase in the COD concentration in the effluent. The loss was due to the formation of pin-point type flocs, not bulking sludge. In an attempt to improve the settling properties a small aeration unit was installed just before the clarifier on 10 November 1981. As this unit did not produce the desired results, it was omitted two weeks later.

As far as denitrification was concerned the unit performed well. Nitrates in the effluent were reduced to concentrations of 0,7 mg/ℓ (as N), which can be regarded as complete denitrification. The average effluent ammonia concentration of 4,6 mg/ℓ (as N) was well below the value required for the general standard.

TABLE 7.3: Average performance of a denitrification unit receiving humus tank effluent when using raw sludge as a carbon and energy source (1981-11-03 - 1981-12-14) (concentrations in mg/ℓ)

	COD (unfiltered)	NH_4^+ (filtered)	NO_2^--NO_3^- (filtered)	PO_4^{3-} (filtered)
Raw sludge	83179	ND	ND	ND
Humus tank effluent	35	3,1	10,8	8,4
Feed entering pilot plant	674	7,7	5,4	ND
Pilot plant effluent	90	4,6	0,7	9,8

ND = not determined.

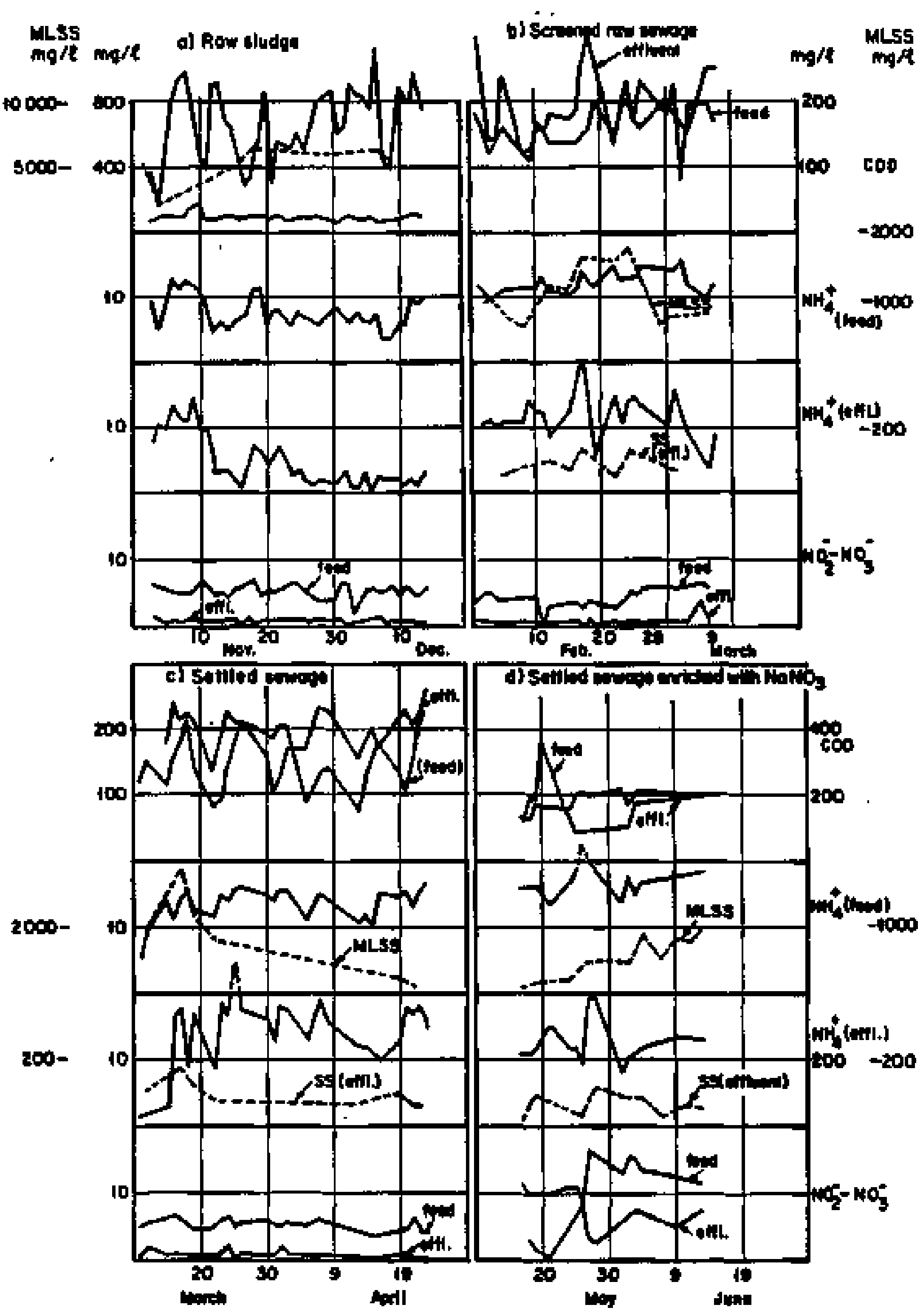


Figure 7.3: Temporal performance of denitrification reactor during different modes of operation.

7.4 Denitrification unit with screened raw sewage as carbon source (1982-01-21 to 1982-03-09)

Raw sludge may not in all cases be available as a possible carbon source for biological denitrification. Therefore, other carbon sources need to be considered in such cases. As an alternative in this study, the use of raw sewage from the Daspoort sewage treatment works was investigated after passage through a screen with 6 by 11 mm openings in order to make it more readily pumpable. This screened raw sewage was then fed in a 1:1 dilution with HTE, resulting in a COD:TKN ratio of 10:1. At the same time the anoxic capacity of the unit was halved and a larger clarifier installed in an attempt to reduce solids carry-over in the final effluent.

The mixed liquor suspended solids concentration decreased to an average of about 1 500 mg/ℓ during the experimental period, while the settling properties of the sludge in the clarifier deteriorated further. Sludge was continuously lost in the effluent, the average suspended solids concentration being about 90 mg/ℓ and consequently the sludge age could no longer be controlled positively and was reduced to an effective average of only about one day.

As it is shown in Table 7.4 and Figure 7.3b, however, the unit performed very well with regard to denitrification. The influent nitrate concentration of about 5,0 mg/ℓ (as N) was reduced to an average of 0,8 mg/ℓ (as N) in the effluent. The high effluent COD values of 182 mg/ℓ on an average were due mainly to the solids carry-over in the effluent.

7.5 Denitrification unit with settled sewage as carbon source (1982-03-10 to 1982-05-05)

During this leg of the study settled sewage instead of screened raw sewage was used as carbon source. This was done in order to investigate the performance of this unit at even lower influent COD concentrations. The lay-out of the denitrification reactor remained unchanged and the mixing ratio with HTE was 1:1.

TABLE 7.4: Average performance of a denitrification unit receiving humus tank effluent when using screened raw sewage as carbon and energy source (1982-01-21 - 1982-03-09) (concentrations in mg/ℓ).

	COD (unfilt.)	TKN (unfilt.)	NH_4^+ (filt.)	NO_2^--NO_3^- (filt.)	PO_4^{3-} (filt.)	TP (unfilt.)	MLSS	VSS
Screened raw sewage	289	29,2	19,4	ND	ND	7,8	ND	ND
Humus tank effluent	45	ND	2,4	9,5	6,9	ND	ND	ND
Feed entering pilot plant	167	ND	10,9	4,8	ND	ND	ND	ND
Denitrification reactor	ND	ND	10,8	0,7	6,6	ND	1465	1330
Pilot plant effluent	182	ND	12,1	0,8	6,3	ND	89	81

ND = Not determined.

TABLE 7.5: Average performance of a denitrification unit receiving humus tank effluent when using settled sewage as carbon and energy source (1982-03-10 - 1982-05-05) (concentrations in mg/ℓ).

	COD (unfilt.)	TKN (unfilt.)	NH_4^+ (filt.)	NO_2^--NO_3^- (filt.)	PO_4^{3-} (filt.)	MLSS	VSS
Settled sewage	230	33,3	21,4	ND	ND	ND	ND
Humus tank effluent	36	ND	2,8	10,8	7,2	ND	ND
Feed entering pilot plant	133	ND	12,1	5,4	ND	ND	ND
Denitrification reactor	ND	ND	14,0	0,7	6,7	1734	1498
Pilot plant effluent	183	ND	13,2	0,7	6,7	118	105

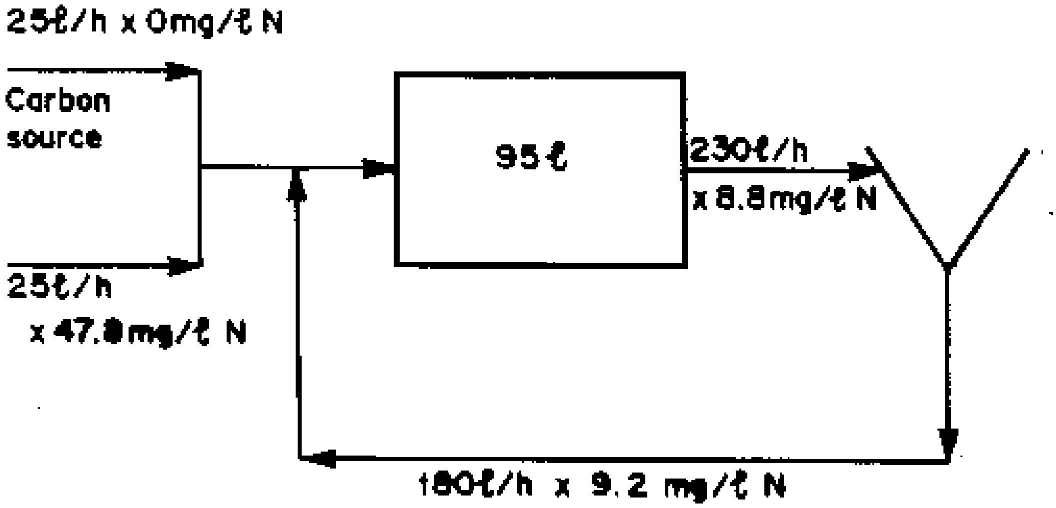
ND = Not determined.

The results in Table 7.6 and Figure 7.3c show that even at feed COD values of as low as 133 mg/ℓ practically complete denitrification was achieved, resulting in effluent nitrate concentrations of 0,7 mg/ℓ (as N). However, the formation of pin-point flocs was still observed and sludge was lost in the effluent as indicated by average suspended solids concentrations of 118 mg/ℓ in the effluent. Therefore, the sludge age was still uncontrolable at about one day. The average MLSS in the reactor during this period was 1 700 mg/ℓ .

By using raw sludge or raw and settled sewage the denitrification capacity of the unit was never exhausted. In order to determine the maximum denitrification rate in the reactor the feed consisting out of settled sewage and HTE mixed in a ratio of 1:1 was spiked with $NaNO_3$ resulting in an average influent nitrate concentration of 24 mg/ℓ (as N). This experiment was carried out over the period from 82-05-06 to 82-06-25. The denitrification capacity of the system was extended to the full by this addition as indicated by an average effluent nitrate concentration of 9,2 mg/ℓ (as N) (Table 7.6, Fig. 7.3d). This net denitrification rate was calculated to be 17 mg NO_3-N per g VSS per h (Figure 7.4). This rate is about 2 to 3 times higher than that reported for the activated sludge process (Simpkins and Gerber, 1981). This rate was achieved at an average MLSS concentration of about 600 mg/ℓ and an effective sludge age of only about one day. Most of the biomass may thus be assumed as having been in a very active state. This is also expressed by the small difference between MLSS and VSS.

7.6 Discussion

The results achieved in these series of experiments indicate that denitrification of humus tank effluent proceeds in a straight forward manner. The most important problem area which should be considered in the design of a slurry denitrification reactor appears to be the loss of suspended solids in the effluent and its effect upon the maintenance of the proper mean cell residence time. In situations where upgrading with respect to phosphorus removal is required as well, this is unlikely to be a point of major concern. In such applications the denitrification reactor would be located prior to the filter but following chemical dosage for phosphate removal, with humus tank effluent being recycled in this point. Figure 7.5 shows just one of several possible arrangements.



(25 x 0) + (25 x 47,8) + (180 x 9,2) - (230 x 8,8) =

0 + 1195 + 1656 - 2024 = 827 mg/h N

Denitrification rate $= \frac{827}{502 \times 95}$ = 17 mg NO_3-N per g VSS per h

Figure 7.4. Calculation of denitrification rate.

TABLE 7.6: Average performance of a denitrification unit receiving humus tank effluent when using nitrate-enriched settled sewage as carbon and energy source (1982-05-06 - 1982-06-25) (concentrations in mg/ℓ)

	COD (unfilt.)	TKN (unfilt.)	NH_4^+ (filt.)	NO_2^--NO_3^- (filt.)	PO_4^{3-} (filt.)	TP (unfilt.)	MLSS	VSS
Settled sewage	345	34,4	24,8	ND	ND	6,9	ND	ND
Humus tank effluent	60	8,0	6,7	6,5 47,6 after spiking	7,0	7,3	ND	ND
Feed entering pilot plant	202	21,2	15,8	24,0	ND	7,0	ND	ND
Denitrification reactor	ND	ND	13,1	8,8	6,8	ND	618	502
Pilot plant effluent	193	21,6	16,3	9,2	6,5	6,9	71	54

ND = Not determined.

Settling properties of the biological sludge can be expected to be much improved under those conditions due to the presence of chemical coagulant. Since the contract for this project was terminated end of June 1982, no experiments with chemical addition could be carried out anymore. But that the assumption is right has been shown by Van Vuuren and co-workers (1981) in applications of the LFB process as well as by results obtained from a full-scale trial of a pre-denitrification unit at Daspoort (Pretoria), where phosphate was removed chemically by pre-treatment with ferric chloride, which was added in concentrations of 120 mg/ℓ as Fe. The lay-out of the full-scale plant is given in Figure 7.6. Typical results are shown in Table 7.7, the values in brackets referring to a control biological filter system. The SVI of the sludge in the denitrification reactor was reported to be about 50 mℓ/g (Thirion, personal communication).

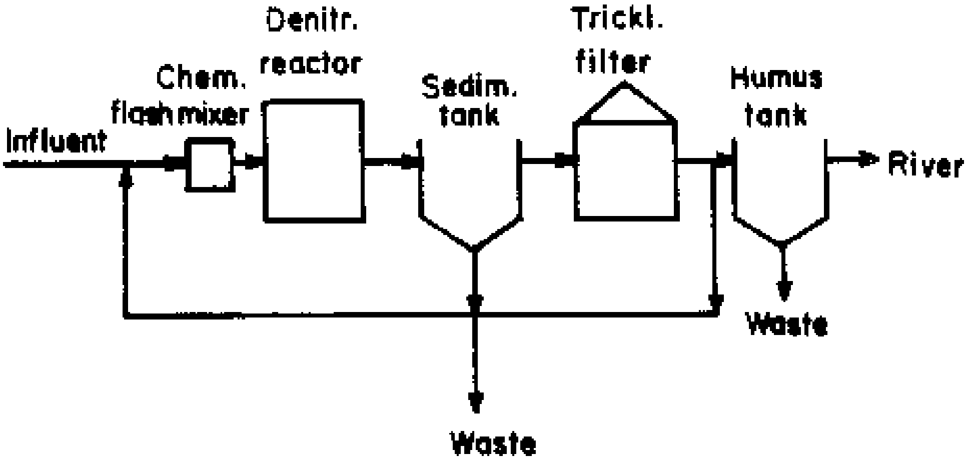


Figure 7.5: Schematic layout of plant for chemical phosphate removal and biological denitrification in trickling filter effluents.

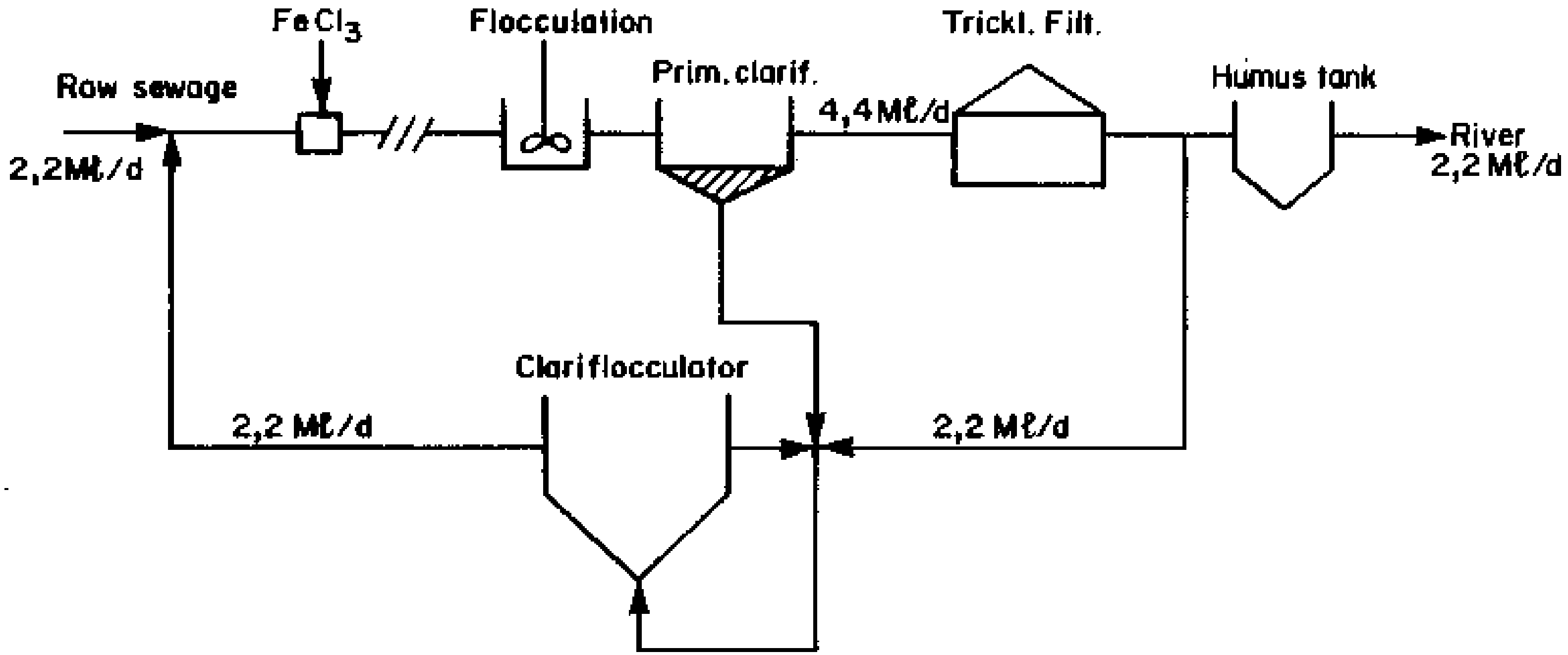


Figure 7.6: Layout of a full-scale pre-denitrification experiment at Daspoort.

TABLE 7.7: Results of full-scale experiments with a pre-denitrification unit using raw sewage as carbon source at Daspoort, Pretoria. (Values in brackets show control values).

Parameter	Mean value (mg/ℓ)
Raw sewage	
COD	406
TKN	31,6
NH_4^+	19,6
PO_4^{3-}	6,5
Biological filter effluent	
COD	60
NH_4^+	1,4
NO_3^-	5,1
PO_4^{3-}	2,0
Reactor effluent	
COD	106
NH_4^+	3,5
NO_3^-	0,4
PO_4^{3-}	6,1
Final effluent	
COD	49(63)
TKN	3,7(7,0)
NH_4^+	1,8(8,0)
NO_3^-	6,4(11,1)
PO_4^{3}	2,4(5,4)

8. CONCLUSIONS AND RECOMMENDATIONS

Based on our findings, obtained during the study on upgrading existing biological filter systems to comply with the effluent standards for nutrient removal, the following, conclusions and recommendations are made:

Orthophosphate can be effectively removed by chemical precipitation. The choice between pre-treatment, post-treatment or split-treatment should be made based on the concentration of condensed phosphates at the point of chemical addition. If high concentrations of condensed phosphates are expected, derived mainly from synthetic laundry detergents post-treatment is recommended.

If only the removal of COD is required, e.g. in the case of an organically overloaded but hydraulically underloaded plant chemical pre-treatment should be considered.

If COD removal as well as orthophosphate removal is required, split-treatment should be considered.

Removal of nitrogenous compounds, especially by the way of biological denitrification should be seen as part of a comprehensive upgrading process comprising facilities for chemical treatment to remove orthophosphate and COD and the possibility for biological denitrification. Based on our pilot plant experiments and full-scale experiments at the Daspoort sewage works a pre-denitrification unit is recommended. Biological filter effluent has to be recycled to the head of the works where it should be mixed with raw or settled sewage. In a following flash-mixer, chemicals will be added to remove orthophosphate and to improve floc-forming in the subsequent denitrification tank. The raw sludge/denitrification solids should be removed in the primary settlers and recycled to the denitrification reactor. The primary settler effluent should then go on to the biological filters. No adverse effect on the biological filters have been observed in practice. The use of raw or settled sewage as external carbon source for denitrification might be preferred to the use of raw sludge. This practice allows all the raw sludge being produced, to be used in the digesters for gas production.

The use of digested sludges might be considered, but due consideration must be given to its higher unbiodegradable COD fraction than that of raw sludge.

Research is required on the full-scale application of pre-biological filter denitrification. Especially the problem of settleability of the sludge and the capacities of the primary settlers to handle the increased flows need further investigation.

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