DRAFT FINAL REPORT

PERFORMANCES OF SEWAGE TREATMENT WORKS DISCHARGING TO AN ESTUARY, A MARINE ENVIRONMENT OR SENSITIVE AREA

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RESTRICTION: This report is restricted to the National Rivers Authority and equivalent bodies in Scotland and Northern Ireland

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CONTENTS

PERFORMANCES OF SEWAGE TREATMENT WORKS DISCHARGING TO AN ESTUARY, A MARINE ENVIRONMENT OR SENSITIVE AREA

A Dee, B Chambers, J Hobson and M Smith

SUMMARY

This report presents the optimum performances of sewage treatment plant that can partially-treat sewage discharging to an estuary or a coastal environment, or provide nutrient removal for discharges to bodies of water considered prone to eutrophication. The National Rivers Authority require such information on which to base consents. The main objectives were to derive indicators of optimum performance expressed as yearly-means or 95 percentiles and to determine a single design loading appropriate to each process. Accordingly loadings and optimum performances are presented for suitable processes comprising: (i) primary sedimentation tanks, (ii) activated sludge plant and oxidation ditches, (iii) conventional biological filtration plant, (iv) rotating biological contactors, (v) tertiary nitrifying filters, (vi) sand filters, and (vii) chemical dosing plant to remove phosphorous. The performances have been determined by collating and evaluating the information, knowledge and experience available within WRc plc. Where possible the collected information has been validated against readily available data from UK sewage treatment works or in its absence from pilot-tests performed at WRc over a period of year in equipment exposed to the external environment. In the event of a lack of such data, information has been abstracted from the literature. **Accordingly the validation data is variable since it has been derived from a number of different sources, and the loadings and optimum performances should only be used for guidance.** Further works' data should be collected, particularly for primary sedimentation tanks. In such cases, works should be visited to verify that the data represent periods when the works were in good operating order.

Report No NR 2808, May 1991
68 Pages, 9 Tables, 9 Figures, 7 Appendices

1. INTRODUCTION

As a consequence of the EC Municipal Wastewater Directive (EC 1991), the National Rivers Authority need performance data on which to base consents for works requiring partial treatment of sewage discharging to an estuary or a marine environment, and for works requiring nutrient removal from effluent discharging to sensitive areas prone to eutrophication. The required data comprises optimum performances and loading rates for the following processes:

- primary sedimentation tanks for SS and BOD removal,

- activated sludge plants and oxidation ditches for BOD removal only, and for nitrification and denitrification,

- conventional biological filtration plant, practising single-filtration or double-filtration for BOD removal and nitrification,

- rotating biological contactors for BOD removal and nitrification,

- tertiary nitrifying filters,

- sand filters for solids removal,

- chemical dosing to remove phosphorous.

Although most of these processes are widely used at sewage treatment works in the UK and much research effort has been expended, the published values for loading and performance vary between authors. For example the 95 percentile performances given for works fitted with biological filters and humus tanks vary between 20:30 (BOD:SS) and 30:45 (BOD:SS) and loadings vary between 0.06 and 0.12 kg BOD/m^3.d. With such conflicting values, none can be used with confidence. Therefore advice is required on the typical performance and loadings available from each process. In particular information is required on the performance of primary sedimentation tanks to partially-treat sewage discharging to coastal waters and processes which remove nitrogen and phosphorus to prevent eutrophication.

Ideally the optimum performances should be validated against data collected from works which have been surveyed to check that they are in good operating order. Given that such visits are not practical within the very short time scale of the project, the suggested peformances have been validated against readily-available data.

2. OVERVIEW OF PROCESS PERFORMANCE REQUIREMENTS

The EC Municipal Wastewater Directive (EC 1991) specifies the level of treatment required for urban waste water. It then defines how treatment can be relaxed in less sensitive areas, such as coastal waters, and how more stringent treatment is required in sensitive areas prone to eutrophication. This section describes the definitions of such areas and gives the levels of treatment currently required. Clearly the current directive must be consulted for a full and adequate description of these two areas.

2.1 Less sensitive areas

This category covers bodies of water which are considered unlikely to become eutrophic because of good water exchange. It includes open bays, estuaries and other coastal waters. Discharges to such areas may require primary treatment comprising the settlement of solids such that the suspended solids and BOD in the effluent are reduced by at least 50 and 20% respectively.

2.2 Sensitive areas

Such areas includes bodies of water which may be prone to eutrophication because of poor water exchange or because they receive large quantities of nutrients. It therefore includes discharges into streams flowing into lakes and direct discharges into lakes. It also covers coastal waters.

The required quality of discharges to such areas is specified in terms of a minimum percentage reduction and an annual mean concentration. At the time of writing, it specifies that for populations greater than 10 000 the minimum percentage reductions of total phosphorous and total nitrogen should be 80 and 70 to 80% respectively, and that the mean concentrations should be as follows:

° 2 mg total P/l and 15 mg total N/l for 10000 - 100 000 pe
° 1 mg total P/l and 10 mg total N/l for >100 000 pe.

3. PROCESS PERFORMANCE STANDARDS

The sewage treatment processes specified in the Introduction are described in the Appendices. Each Appendix pertains to a particular process, describing how treatment occurs, outlining the type of equipment used and listing the main process constraints that affect effluent quality. The performances and loadings presented are confirmed where appropriate by comparison with plant performance data.

The data, presented in the Appendices, have been used to derive the summary of optimum performances and loadings, presented in Table 3.1. Conventional expressions have been used for each process loading. **Accordingly for primary sedimentation tanks and tertiary sand filters, loadings are based on the maximum flow to treatment, while those for the other five processes are based on the average daily flow.**

With the exception of chemical treatment of phosphorous, each of the processes covered in the report are dependent on either the action of micro-organisms degrading organic matter or on the nature of the sewage solids in terms of their particle size or flocculant nature. Such processes are complex and can be affected by a range of operating conditions, such as temperature or the quality of the influent, that are outside of the operator's control. For the general climatic conditions prevalent in the United Kingdom and for the typical mix of domestic sewage and industrial effluent treated at works, each process should achieve the standard performance given in Table 3.1. **But in particular instances where disruptions occur in the treatment process, the standard performances may not be realised in practice.**

Each process operates optimally within well-defined operating limits. Operation outside of these constraints will cause performance to be impaired. The limits for each process are described in the Appendices. Additonal features are listed below:

1. For primary sedimentation tanks, data from works treating dilute crude sewages e.g. <300 mg/l SS indicate that 50% solids removals are not always achieved in practice. Accordingly facilities for dosing

coagulants or flocculants may be required to enhance removals. To determine if such equipment is required settlement tests will need to be performed.

2. Activated sludge plant operate optimally when designed to produce either a nitrified or a non-nitrified effluent. **Partial-nitrification is not recommended, when the plant's operation is liable to be unstable.**

3. The performance of biological filters is generally not as good as activated sludge plant since they produce higher effluent suspended solids.

4. Rotating biological contactors are normally installed at small works and accordingly reliable process data is lacking. Table 3.1 presents performances and loadings that should be achievable in practice.

5. The level of BOD in influent supplied to tertiary nitrifying filters should be below about 30 mg/l to ensure optimal filter performance.

6. The performance of tertiary sand filters is affected by the influent suspended solids level. When such filters treat effluents from biological filter plant, which have higher suspended solids levels than those from nitrifying activated sludge plant, their performance deteriorate and 95-percentile effluent qualities of 15 mg/l SS should be achieved.

7. Chemical dosing requires an adequate control system to minimise the quantity of chemical precipitant passing into the water course.

Table 3.1 - Process performance

Process	Loading	Performance Mean	Performance 95%ile
		BOD:SS:AmN	
1. Primary sedimentation tanks	1.2 to 1.4 m/h at the maximum flow to treatment.	40 to 80% SS removal. 10 to 50% BOD removal	-
2. Activated sludge plant:	Sludge loading (d^{-1}):		
fully-nitrifying	0.08 - 0.11	-	15:20:5
non-nitrifying	0.35 - 0.5	-	20:30:-
oxidation ditch	0.05 - 0.08	-	15:20:5
anoxic zones		50% NO_3-N removal	
3. Biological filters:	$m^3/m^3.d$ kg BOD/m^3.d		
single filtration	0.6 0.09	-	30:45:15
double filtration	0.6 0.09	-	20:30:10
4. RBC:	Surface loading (g BOD/m^2.d)		
nitrifying	2 - 3	-	30:45:15
non-nitrifying	3 - 5	-	30:45
5. Tertiary nitrifying filters	60 gN/m^3.d	-	-:-:5
6. Tertiary sand filters treating effluent from a nitrifying activated sludge plant:	Irrigation velocity at maximum flow to treatment ($m^3/m^2.d$)		
downward flow	250	-	-:10:-
upward flow	400	-	-:10:-
7. Chemical treatment	As prescribed in Appendix G	2 - 3 mg P/l 1 - 2 mg P/l 0.5 - 1 mg P/l	-

Note: All loading except those noted are based on average daily flow.

4. CONCLUSIONS

Optimum performances and loading values have been presented for sewage treatment plant that discharges to an estuary or a marine environment.

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EC (1991) Proposal for a Council Directive concerning municipal waste water. 4460/91, ENV 35, February 1991.

APPENDIX A - PRIMARY SEDIMENTATION TANKS

A1 PROCESS DESCRIPTION

Primary sedimentation tanks provide quiescent conditions necessary to clarify crude sewage. Sedimentation occurs through discrete particles which settle at a constant velocity and through flocculation where non-settleable solids agglomerate with other particles to form settleable solids.

The design of sedimentation tanks is described in the Manual of British Practice (Institute of Water and Environmental Management 1973). The most commonly-used designs pertain to either circular or rectangular tanks fitted with a scraper assembly. In circular tanks crude sewage enters at the centre usually through a circular stilling box and then flows radially towards the periphery at a sufficiently low velocity to allow solids to settle to the tank floor. Slowly rotating scrapers plough the settled sludge to a central outlet hopper and clarified sewage overflows a peripheral weir. In rectangular tanks the sewage flows longitudinally and the scrapers plough the settled sludge to the outlet hopper located at the end of the tank receiving influent.

A2 THEORY

In 1904 Hazen (1904) proposed the concept of discrete particles settling in an ideal tank under quiescent conditions and showed theoretically that such particles will only reach the tank floor if their settling velocity exceeds the ratio of the flowrate to plan area of the tank. This ratio is termed the tank upward flow velocity. Modern practice is to design on an upward flow velocity of 1.2 to 1.4 m/h at the maximum flow to treatment.

White and Allos (1976) performed classical settlement tests on sewage from Davyhulme sewage treatment works in 1 m tall columns fitted with tapping points at different depths. They found that the concentration of solids at any given time interval only changed slightly with depth except near the bottom of the column. This is in marked contrast to what would be expected from discrete particles, the concentration of which should increase with depth. The explanation is that the solids flocculate progressively with time and then settle. They found that most of the settlement occurred within a period of 2 hours. Modern practice is to design on a minimum retention time of about 2 hours at the maximum flow to treatment.

Settling tests were performed at the Water Pollution Research Laboratory (Ministry of Technology 1965) on crude sewage from four different works in columns having the same depth as a typical primary sedimentation tank. The results presented in Figure A1 indicate that, as the concentration of solids in the crude sewage rises from 260 to 930 mg/l, their removal increases from 65 to 80%. The increased removals are explained by flocculation increasing with concentration.

Quiescent settling tests were performed in columns at the Water Pollution Control Laboratory (Ministry of Technology 1965) to compare the settlement of a mixture of humus sludge and sewage, and sewage only. The results indicate that though the addition of humus sludge increased the initial solids concentration from 330 mg/l to 1020 mg/l, the solids concentration in the clarified effluent only increased from 170 mg/l to 180 mg/l. Similar results were obtained from tests performed using activated sludge. Therefore the return of secondary sludge only marginally increases the concentration of solids in tank effluent.

Other factors can affect performance. Price and Clements (Price and Clements 1974) have shown that wind speed and density gradients or thermal currents cause settled sewage quality to deteriorate.

A3 PREDICTED AND ACTUAL PERFORMANCE

Given that effluent quality is affected significantly by the concentration of suspended solids and BOD in the crude sewage, tank performance is normally expressed as the percentage removal of suspended solids and BOD from the influent excluding any returns of secondary sludge or liquors from on-site dewatering. Since most tanks are designed to give maximum solids removal, upward flow velocity is a less significant variable when examining works data.

Given that primary sedimentation tanks will be installed at coastal sites for partial treatment, performance data is ideally required from works which do not return secondary sludges to primary treatment. Price and Clements (1974) have performed studies on full-scale tanks treating crude sewage only. Their results presented in Table A1 indicate that though solids removals of 72% were achieved by tanks treating crude sewage with mean solids concentrations above 300 mg/l SS, removal efficiency dropped to 38% for a tank treating crude sewage with a solids concentration of about 200 mg/l SS.

Table A1 - Solids removal efficiencies determined for primary sedimentation tanks treating crude sewage (Price 1974)

	Number of samples	Tank dimensions (m)			Mean upward flow velocity (m/h)	Mean SS (mg/l)		Percentage removal
		Length	Breadth	Mean depth		Infl.	Effl.	
Macclesfield	9	34.8	10.6	2.1	0.32	219	137	37.6
Nottingham	11	91.5	34.2	2.0	0.50	491	157	68.0
Rotherham	15	93.0	12.2	1.9	0.40	401	113	71.8

The tests of Price and Clements were performed in rectangular tanks only and therefore performance data is required for circular tanks. Lockyear (1980) surveyed data from 163 works and found that while the suspended solids and BODs in settled sewage from rectangular tanks averaged 129 and 170 mg/l, the respective levels in cicular tanks were 112 and 157 mg/l. Since the mean upward flow velocity in the rectangular tanks was not significantly different to that in circular tanks, the results suggest that circular tanks are more efficient.

Additional data is available from a large number of sites covering mainly circular tanks treating returns of activated and humus sludge. The results presented in Figure A2 indicate that a significant fraction of sites have suspended solids and BOD removal efficiencies below 50% and 20% respectively. Anderson (1981) has collected extensive daily performance data covering the settlement of crude sewage, return sludges and liquors at works containing secondary treatment. The results indicate that though removal efficiencies are above 50% at influent solids concentration above 300 mg/l solids, they can drop below 50% at concentrations in the vicinity of 200 mg/l. Therefore primary sedimentation may be insufficient in some cases, particularly where sites receive dilute crude sewage containing 200 mg/l SS.

In the event that removal efficiencies are not met, settlement will need to be enhanced by adding coagulants and flocculants. Matthews (1977) has indicated that dosing crude sewage with coagulants such as lime, iron salts, aluminium salts or polyelectrolytes can reduce the level of suspended solids in settled sewage to about 60 mg/l thereby ensuring that removal efficiencies of 50% can be achieved in practice. However operating costs will be higher and the volume of sludge requiring disposal will be increased.

A4 OPERATING CONSTRAINTS

The performance of primary sedimentation tanks is maximised by ensuring the following operating conditions:

- sludge is withdrawn regularly and is not allowed to accumulate in any tank,
- septic sludges are not imported to site to mix with the crude sewage entering the tanks.

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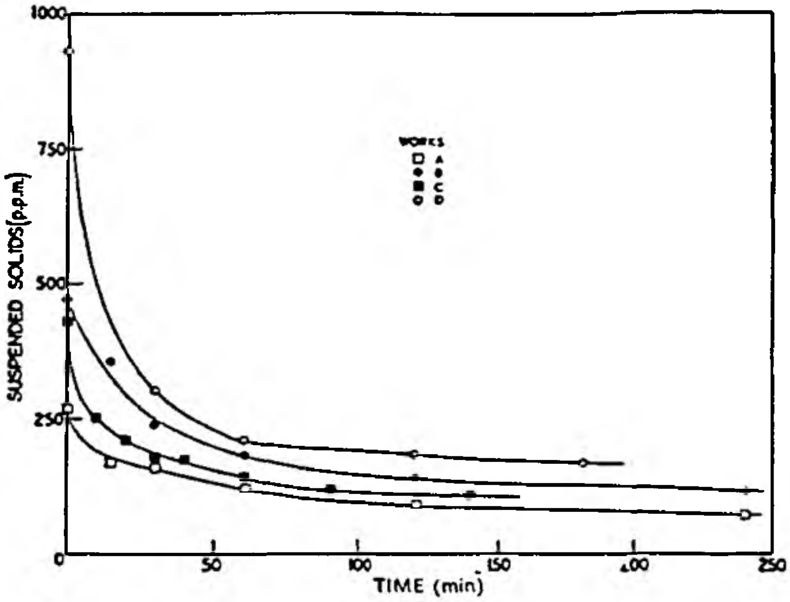

Figure A1 - Settling properties of suspended solids in samples of crude sewage from four sewage treatment works

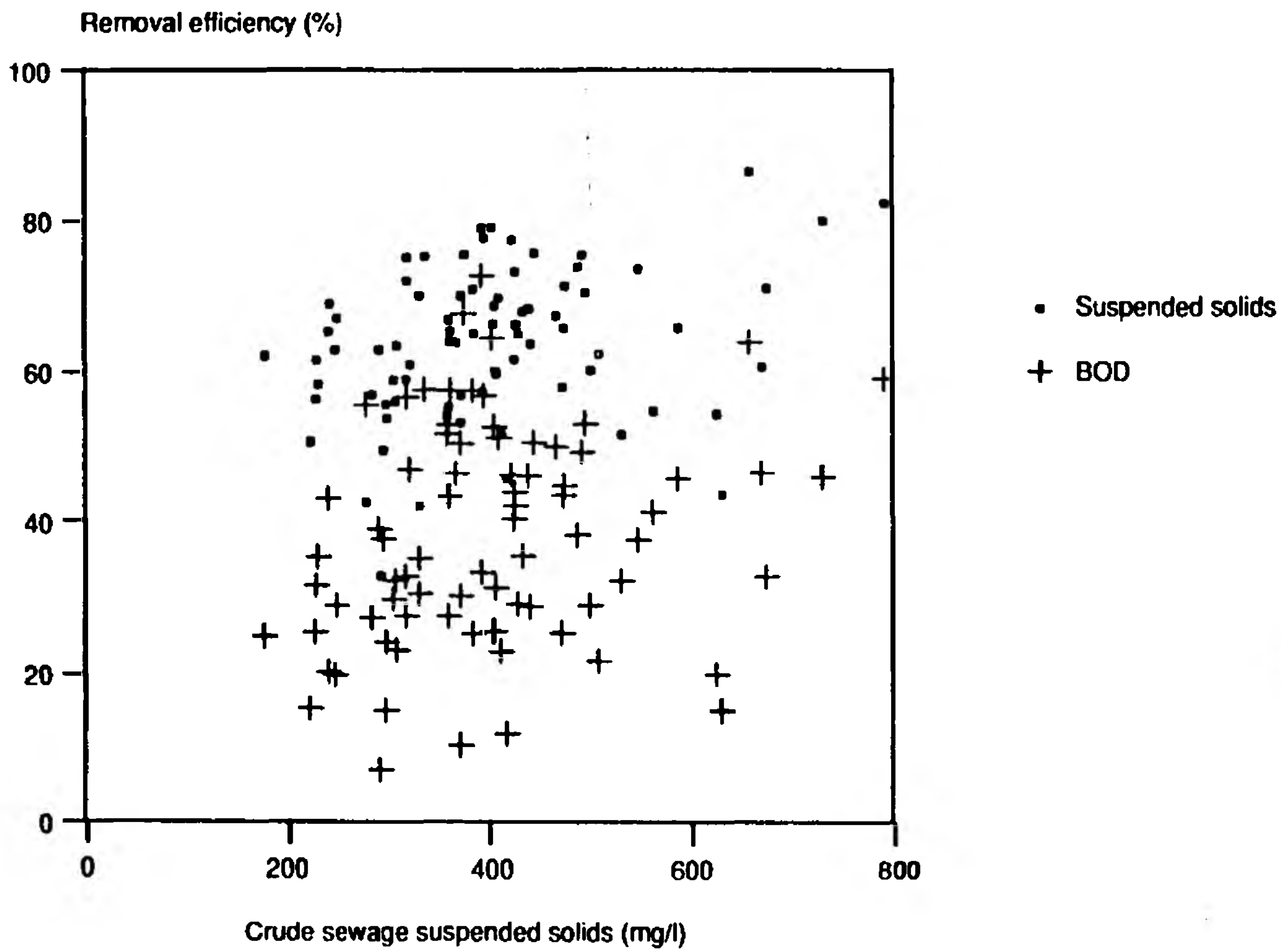

Figure A2 - Primary sedimentation tanks removal efficiency

APPENDIX B - ACTIVATED SLUDGE PLANT AND OXIDATION DITCHES

B1 PROCESS DESCRIPTION

All variants of the activated sludge process consist of two stages connected in series. The first stage is an aeration tank which can be considered as a biochemical reactor and the second stage is a separation process which is designed using the principles of gravity sedimentation. Efficient performance of both stages of the activated sludge system is vital and the interconnection between the two by which sludge is recycled cannot be ignored in any detailed analysis of the overall process.

The aeration tank contains an aerobic suspension of various micro-organisms which, given the right conditions, can produce the required degree of treatment. The sewage to be treated enters the tank and is intimately mixed with the microbial suspension and supplied with oxygen. Organic material in the sewage is oxidised and there is a corresponding growth of microbial biomass. The biomass is separated from the treated effluent in the settling tank and the thickened sludge is recycled back to the aeration tank inlet as shown in Figure B1. Effluent is discharged from the final tank for further treatment or to the receiving watercourse.

A small proportion of the recycled flow is removed from the system as surplus sludge. This fraction is an important process variable and its value not only determines the average concentration of solids in the aeration tank but also the overall rate of treatment.

An important process variant is the oxidation ditch which differs from the conventional activated sludge process with respect to the aeration tank configuration. In conventional systems the influent sewage enters the aeration tank and receives treatment before leaving the tank for final settling. The design of the oxidation ditch aeration tank is such that the tank contents are recirculated in a well-defined manner by the action of the aeration system. Oxidation ditches are usually, but not always, installed to treat crude sewage which has not undergone the process of primary sedimentation.

B2 THEORY

The extreme complexity of the microbial reactions which occur in activated sludge processes has prevented the development of a comprehensive theoretical description of plant performance. Plant design is based on a combination of simplified theory and empiricism derived from previous experience of systems which gave good results.

The main problems of activated sludge plant design are specifying the dimensions of the aeration and final settling tanks and providing an adequate amount of process oxygen, all such that effluent of the required quality is produced. Provision must also be made for the effects of time-varying influent sewage characteristics of a daily and seasonal nature.

Traditionally activated sludge plants were designed on the basis of BOD removal in the aeration tank and suspended solids removal in the final settling tank. More recently, requirements to achieve effluent quality standards which include limits on ammonia concentration have been imposed. Hence the concentration of ammonia in influent sewage has become an important design variable.

Plant effluent quality and oxygen requirements are broadly related to gross design parameters such as sludge loading rate and nominal sewage retention time. Nitrification has been found to be related to the growth rate of specific nitrifying organisms. Plants designed to remove ammonia are based on principles which relate microbial growth rate to simple operating parameters.

Recently developed design methods attempt to relate plant inputs to effluent quality by means of materials balances performed about the process. Successful process models are usually complex and computer methods are required for solution (Chambers and Jones (1988).

Oxidation ditch systems have not received the same degree of theoretical investigation as conventionally constructed plants. It is standard practice to design oxidation ditch plants on a purely empirical basis using conservative dimensioning criteria.

B3 PREDICTED AND ACTUAL PERFORMANCE

In general terms the performance requirements of activated sludge plants can be divided into the production of fully-nitrified effluent and the provision of carbonaceous treatment only. In the former case, effluent consent limits would be defined in terms of BOD, SS and ammonia concentrations, in the latter case, BOD and SS only. It is common practice to define effluent quality in terms of 95%-ile values. Operating experience has shown that the average concentration of parameters measured in plant effluent is about half the 95%-ile value. Thus an activated sludge plant might be required to produce effluent quality of 15:20:5 mg/l BOD:SS:NH_3-N respectively on a 95%-ile basis. Average values corresponding to this standard would then be 7.5, 10 and 2.5 mg/l approximately.

It should not be assumed that activated sludge plants can be designed to produce every degree of effluent quality. Some relevant points are summarised below in Table B1 and the explanatory notes which follow.

Table B1 - Summary of typical effluent qualities required from activated sludge plants

Concentration (mg/l)						
95%-ile			Average			Explanatory Note
BOD	SS	NH_3-N	BOD	SS	NH_3-N	
10	10	2	5	5	1	1
15	20	5	7.5	10	2.5	2
15	20	10	7.5	10	5	3
20	30	-	10	15	-	4
40	60	-	20	30	-	5

1) There are some activated sludge plants in existence which regularly achieve effluent quality of this standard. However, it must be emphasised that such plants were not usually designed for this level of performance. BOD and suspended solids concentrations of this magnitude

suggest that tertiary treatment is necessary. **Effluent ammonia concentrations of less than about 5 mg/l on a 95%-ile basis are very difficult and expensive to achieve with certainty.** There is no history of plant design in the UK which covers this level of effluent quality. 2-stage activated sludge plants or plants with very efficient flow balancing would probably be required.

2) These effluent concentrations are normally achieved by a well-designed, fully-nitrifying activated sludge plant which treats a typical mixture of domestic and industrial wastewater. Many plant suppliers will not normally offer process guarantees for effluent concentrations less than these values. A high proportion of industrial waste may result in the inhibition of nitrification. It is also important to assess the effect of very severe winter temperatures on the rate of nitrification. A sewage temperature of less than 5 °C will present design difficulties for a plant required to nitrify to this level. It is also possible to combine such effluent quality with the use of anoxic zones for up to 50% nitrate reduction by denitrification.

3) Care should be exercised when attempting to design plants to achieve intermediate levels of nitrification. The rate of nitrification is very sensitive to slight changes in several design variables and **it is virtually impossible to operate a plant to achieve a degree of nitrification which is not substantially complete.** Process instability can occur if microbial population shifts are continually taking place. Foaming, bulking and denitrification problems may result. **It is recommended that plants are designed either to achieve complete nitrification, as defined previously, or are not designed for nitrification at all.**

4) Activated sludge systems can be designed to achieve BOD and suspended solids removal without ammonia oxidation. However, during periods of high ambient temperatures it has been observed that many such plants achieve a partial degree of nitrification. If the sewage temperature approaches 20 °C then substantial nitrification can occur at sludge ages

of less than 5 days. Unwanted nitrification can result in denitrification in final settling tanks and consequent solids loss.

It can be operationally difficult to suppress nitrification completely. It may be necessary to reduce MLSS concentrations to less than 2000 mg/l and operate at a fairly high rate. Sludge production can be high and surplus sludge dewatering characteristics may be poor.

5) **Activated sludge plants cannot be designed to produce this type of effluent quality.** Existing systems which achieve this level of performance are invariably overloaded or badly designed. The requirement to produce this level of treatment is becoming less common, especially at large sites.

It is recommended that the use of activated sludge processes is not considered for such schemes. It is relatively simple to design for better quality effluent without incurring any identifiable cost penalty and hence the installation of activated sludge plants for partial treatment can never be justified.

The available evidence suggests that activated sludge processes should be either designed to nitrify completely (Note 2, Table B1) or designed for carbonaceous treatment only (Note 4, Table B1). Operating experience has shown that well-designed activated sludge plants are a very cost-effective process for achieving good quality effluents. In some cases it can be shown that fully-nitrifying systems exhibit similar operating costs to carbonaceous treatment systems if sludge disposal costs are included in the comparison.

Typical values of design parameters used to predict plant performance are given in Table B2 for fully nitrifying and non-nitrifying plants as described in Table B1. Oxidation ditches are usually designed to produce fully-nitrified effluents.

Table B2 - Design parameters used to predict plant performance

	Sewage retention time (h)	Sludge loading rate (d^{-1})	Sludge age (d)	Sludge production rate (kg/kg)
Fully-nitrifying	9 - 12	0.08 - 0.11	12 - 15	0.8 - 1.0
Non-nitrifyng	3 - 5	0.35 - 0.5	4 - 6	0.9 - 1.2
Oxidation Ditch	16 - 24	0.05 - 0.08	15 - 20	1.0 - 1.2

Table B2 is derived with the following assumptions:

° The influent sewage is a 'typical' mixture of domestic and industrial wastewater with no components inhibitory to nitrification.

° The ratio of BOD to ammonia concentration is typical for sewages as described above.

° Oxidation ditches treat crude sewage.

° Process volumes include anoxic zones when appropriate.

° All values are averages.

B4 OPERATING CONSTRAINTS

Conventional designs of activated sludge process can be installed to achieve complete nitrification as defined previously. The incorporation of anoxic zones can result in removal of about 50% of the nitrate nitrogen produced by ammonia oxidation. Conventional processes cannot be easily modified to achieve complete nutrient removal (N & P) by biological means.

The presence of toxic components from industrial discharges is known to inhibit nitrification. Easily biodgradeable trade effluents can result in oxygen transfer limitations. In some cases the oxygen demand can only be satisfied by the use of pure oxygen instead of air.

Industrial discharges and poor process design can also result in poor sludge settleability. This phenomenon, known as 'bulking' affects the performance of the final settling stage and can cause complete process failure in severe cases.

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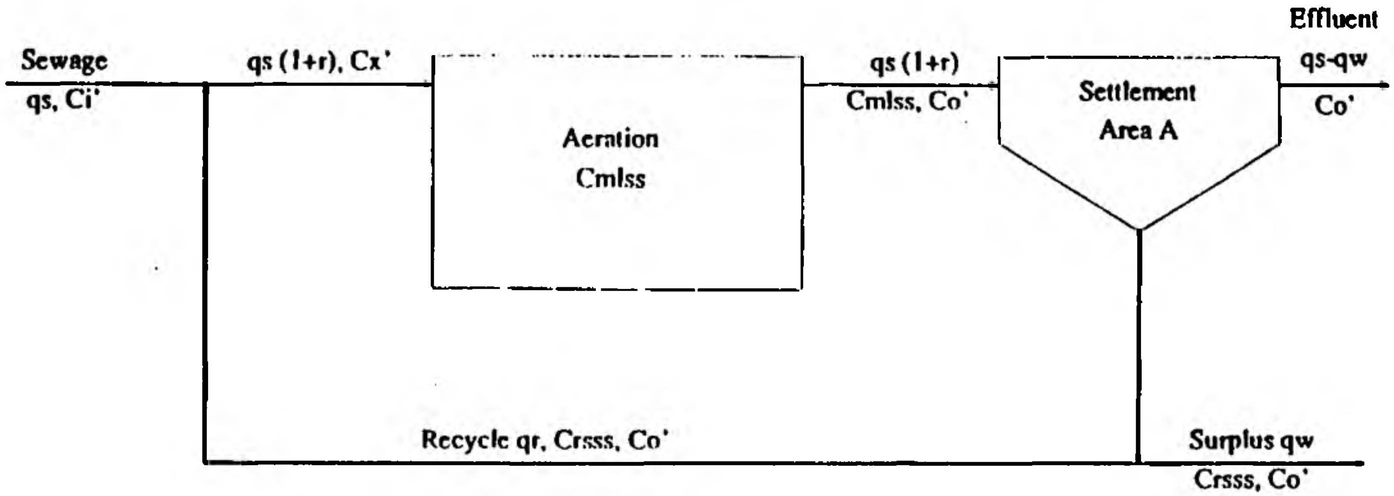

qs = average sewage flow

qr = recycle flow

r = recycle ratio, qr/qs

C' = concentration of organic contaminants in sewage

Cx' = (Ci' + r Co')/(1 + r)

Figure B.1 - Activated sludge flowsheet

APPENDIX C - CONVENTIONAL BIOLOGICAL FILTER

C1 PROCESS DESCRIPTION

Conventional biological filtration plant are used to oxidise BOD and ammoniacal-nitrogen and reduce suspended solids in effluent from primary sedimentation tanks. Treatment occurs through sewage trickling over the surface of medium covered in carbonaceous and nitrifying bacteria, which remove BOD and ammonia respectively. Carbonaceous bacteria predominate, particularly in the top of such filters where BOD concentrations are highest, because they have higher growth rates than the nitrifiers. Nitrification mainly occurs at the bottom of such filters, where the lower BOD concentrations reduce the extent of carbonaceous bacteria (facultative heterotrophs) and allow the population of nitrifiers (autotrophic bacteria) to increase. Air to oxidise the pollutants ventilates by natural convection through the spaces between the medium which also form the passages for the flow of effluent receiving treatment. The filter effluent, which contains debris such as detached film, larvae and worms, gravitates to humus tanks for clarification.

Typically conventional biological filters are about 1.83 m deep and contain granite or slag medium of about 40 or 50 mm size. The medium should comply with the British Standard 1438 (British Standards Institution 1977), which specifies that medium should have a uniform size, a spherical shape and an irregular surface texture for optimum performance. The effluent is dispersed over the top of the medium through a rotating distributor and treated effluent drains through the elements of medium to a sloping floor, where drain tiles allow unobstructed flow to the outlet. The humus tanks are similar in construction to primary sedimentation tanks and modern designs invariably have scrapers.

The various modes of operation, which are used for conventional filters, include single filtration and double filtration. Single filtration comprises a single stage of biological filtration followed by clarification in humus tanks. Double filtration uses two stages of treatment with additional clarification between the stages. In such a filtration mode, the first stage usually operates at a high organic loading and has large media e.g. 63 mm size to accommodate the increased film growth. The second stage has small media e.g. 28 mm size to provide a high surface area of biofilm for removal of pollutant enhancing treatment.

C2 THEORY

The steps by which organic pollutant is removed from sewage in biological filtration plant comprise:

(i) transport of pollutant from the bulk of the liquid film to the surface of the biofilm,

(ii) adsorption of insoluble components,

(iii) diffusion of soluble components into the biofilm and then

(iv) oxidation of the organic components by bacteria.

Such removal processes are by their nature complex.

Hoyland and Harwood (1980) developed a model for carbonaceous removal in biological filters during the early 1980's. He assumed that the removal of BOD was limited by the diffusion into the biofilm and calibrated the model against data from about six sewage treatment works. The relationship between influent and effluent BOD and hydraulic loading predicted by the model for 50 mm medium is presented in Figure C1. It indicates that effluent BOD is determined largely by hydraulic loading and to a lesser extent by influent BOD. It serves to demonstrate that when specifying filter performance, both hydraulic and organic loading should be given.

The nitrification processes that occur in such filters follow the same mechanisms described for tertiary nitrifying filters, though performance is reduced because of competition from carbonaceous bacteria. Accordingly Appendix E covering tertiary nitrifying filters should be consulted for a description of nitrification theory.

C3 PREDICTED AND ACTUAL PERFORMANCE

Removal rates in biological filters are related to the surface area of the medium and hence their size. Accordingly the most suitable loading criterion should be based on the surface area of the medium. Given that conventional filters contain standard sizes of medium, volumetric loading criteria are normally adopted however.

Modern design practice for single-stage filters comprises the use of 40 or 50 mm size slag or granite medium and operation at hydraulic loadings of about 0.6 m^3/m^3d equivalent to an organic loading of about 0.09 kg BOD/m^3d. Such filters should produce a 95 percentile effluent quality of 30:45:15 (BOD:SS:AmN).

Double filtration units which can be designed to operate at similar overall loadings, should achieve a 95 percentile effluent quality of 20:30:10. In such filters, the first stage, which contains a large size of medium e.g. 63 mm size, can be operated at organic loadings of 0.25 kg BOD/m^3d. The second stage, which contains a small size e.g. 28 mm medium is normally designed to provide the balance of capacity necessary to meet the overall loading of 0.09 kg $BOD/m^3.d$ specified for both units.

The humus tanks are designed on an upward flow velocity of 1.2 to 1.4 m/h at the maximum flow to treatment, which is the same as used for primary sedimentation tanks. At such a velocity the readily-settleable solids generated by the filter are mostly removed but the colloidal material passes through in the effluent.

Pilot scale tests were performed at the Water Pollution Research Laboratory in the mid 1970's to determine the performance available from single-stage filtration but the results were not published. Seven filters, each about 2.4 m long, 2.1 m wide and 1.83 m deep, were operated in parallel treating settled domestic sewage having a mean BOD of 305 mg/l. The results presented in Figure C2 compare the actual yearly-average performances of the filters with effluent BOD's predicted by Hoyland's model. To meet a 95 percentile of 30 mg/l (equivalent to an average effluent BOD of 15 mg/l at a typical value of two for the ratio of the 95 percentile effluent BOD to the mean), Figure C2 indicates that the maximium applied hydraulic loading is about 0.6 m^3/m^3d. Given that the performance of filters is less sensitive to BOD concentration than hydraulic loading and that most filters treat an average influent BOD of 150 mg/l, the maximum hydraulic loading pertains to an organic loading of 0.09 kg BOD/m^3d.

The results presented in Figure C2 indicate additionally that the second-stage units used in double filtration plant which contain small sizes of medium such as 28 mm, can achieve an effluent BOD of about 8 mg/l at a loading of 0.6 m^3/m^3d. Given a factor of two, such units should therefore meet a 95 percentile effluent BOD of 20 mg/l.

The foregoing relates to work done on one domestic sewage (i.e. no industrial component). Given that performances predicted by Hoyland's model were validated against data from plants treating brewery waste and effluent from heavy engineering, it ties in well with UK practice and experience.

C4 OPERATING CONSTRAINTS

The performance of filters can be impaired by excessive growths of bacteria, fungi or algae in their top surface. Such growths are promoted by sewages of high organic strength e.g. above 300 mg/l BOD and particularly by high concentrations of readily biodegradable carbohydrates such as dairy wastes. Such accumulations can be reduced by recirculating effluent which lowers the strength of the influent BOD or by decreasing the rotational speed of the distributor to about 10 revolutions/hour which promotes sloughing from the filter.

Filter performance particularly nitrification, can be inhibited by toxic discharges from industrial sites.

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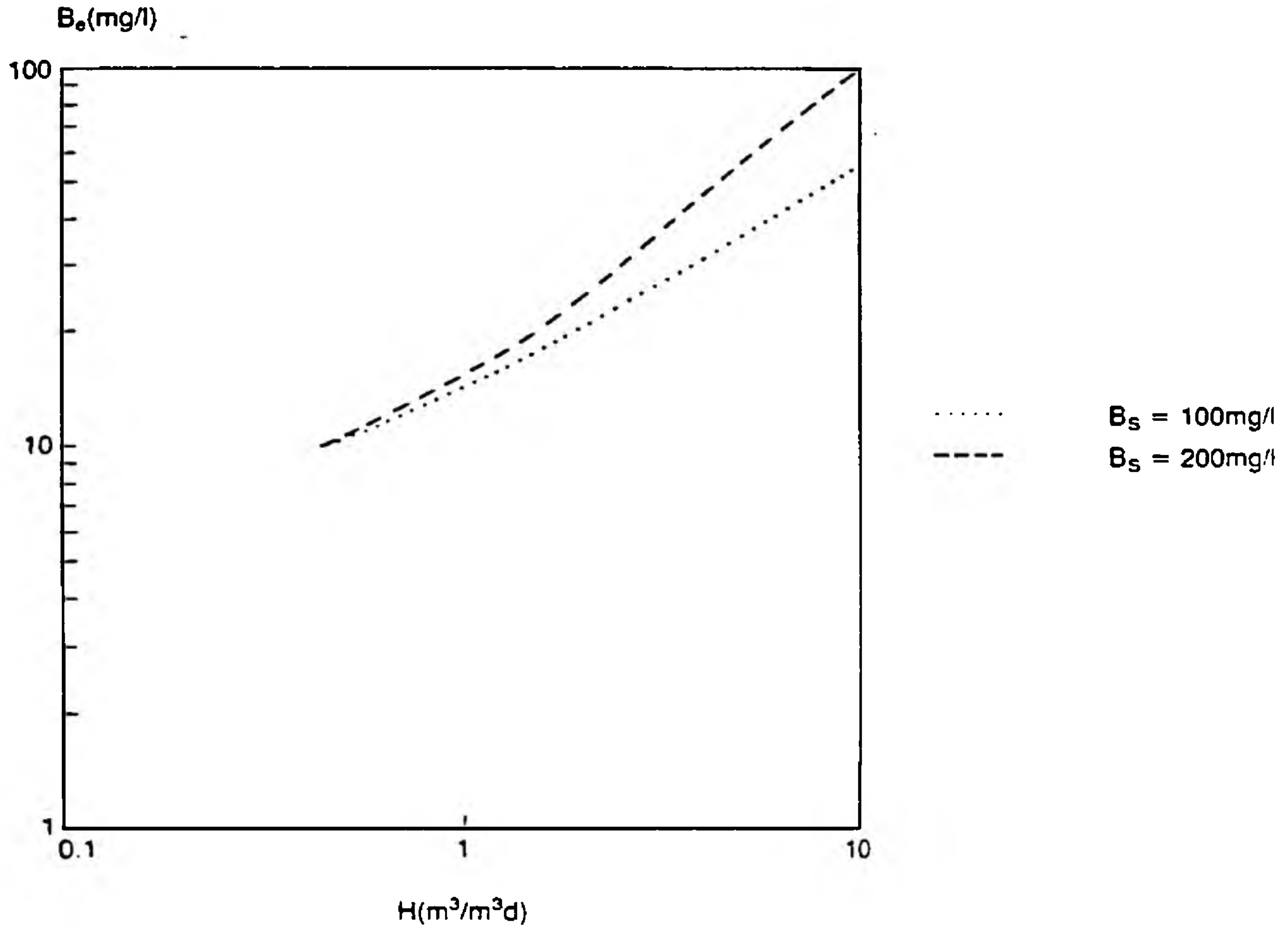
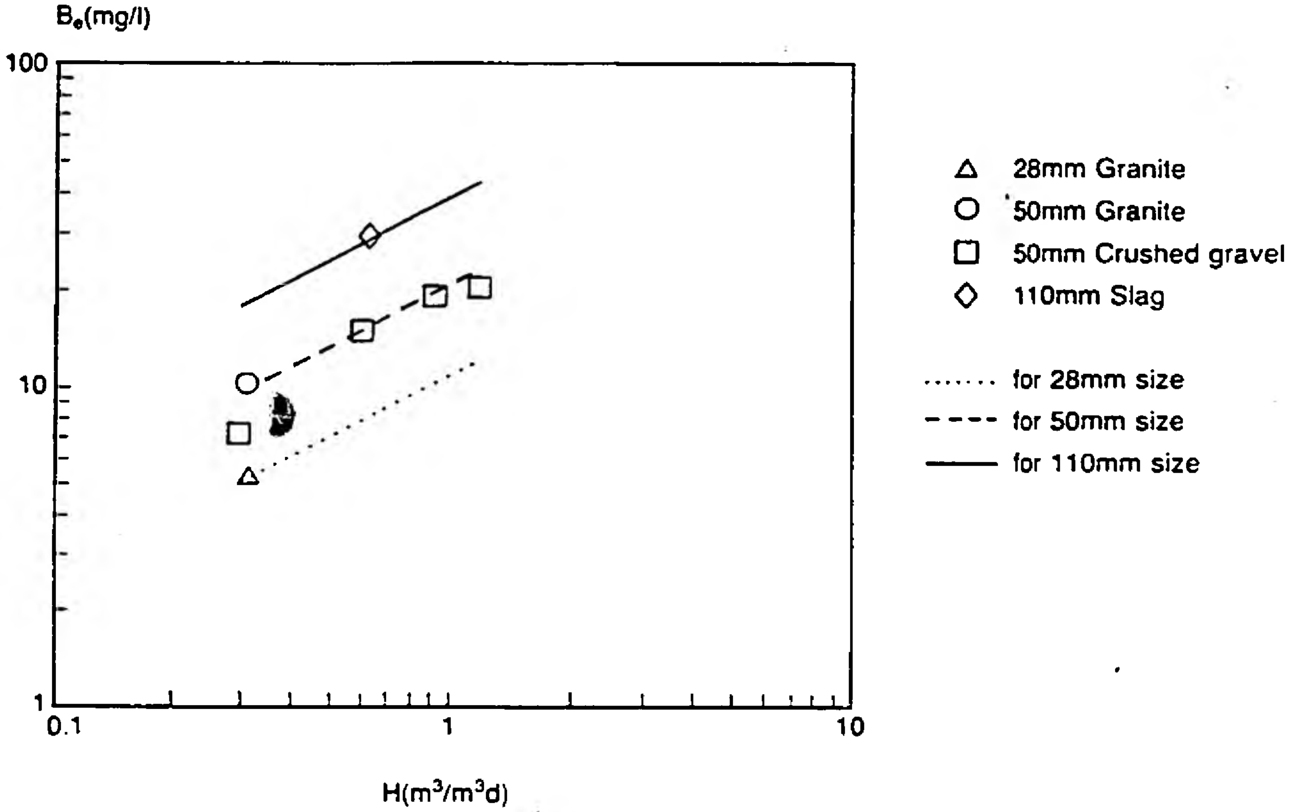


NOTES:

B_s : settled sewage BOD (mg/l)
B_e : settled filter effluent BOD (mg/l)
H : hydraulic loading (m^3/m^3.d)

Figure C.1 - BOD performance predicted by carbonaceous filter model of Hoyland



Each filter . 2.4m long, 2.1m wide and 1.83m deep, treating settled domestic sewage of mean BOD 305mg/l

NOTES:

B_e : settled filter effluent BOD (mg/l)

H : hydraulic loading ($m^3/m^3.d$)

Figure C.2 - Comparison of the actual yearly-average performance with that predicted by the model of Hoyland

APPENDIX D - ROTATING BIOLOGICAL CONTACTOR (RBC)

D1 PROCESS DESCRIPTION

In the initial configuration of this process, parallel discs on a shaft, submerged to about 40% of their depth in a tank into which settled wastewater is fed, are slowly rotated. Biomass grows on the discs and so alternately contacts wastewater and oxygen in the air.

Later designs started to use fluted or ribbed discs, to increase surface area. More modern designs more frequently use rotors of complex geometry, often resembling the media used in high-rate biological filters, or occasionally even the random media also used in that process, packed into a cage.

In the UK the process has been used almost exclusively for small works, ranging from single household up to 1000 population equivalent. Although as described, the process is simply an alternative biological treatment process to e.g. the biological filter, it is frequently marketed as a package, usually enclosed, with integral primary and secondary settling zones. This aspect has made it popular with hotels, restaurants and campsites, as well as the water industry.

Treatment of the wastewater takes place via biological oxidation and biological flocculation, with subsequent removal of biological solids. The process is an alternative geometry of a fixed film reactor and is in many other respects similar to biological filtration. Its benefits are (i) the lack of need for a flow distribution device and (ii) a greater treatment rate for an equivalent surface area of biofilm as compared to biological filtration.

D2 THEORY

Organic pollutants and nutrients from wastewater are absorbed into the biofilm during the submerged phase of the rotation cycle. Oxygen is absorbed primarily during the aerial phase, completing the conditions necessary for purification. Surplus growth of biomass shears from the disc, leaves the biozone with the treated wastewater and requires removal in a secondary settlement zone or other solids removal device.

Several mathematical models of the process have been developed, based around biofilm kinetics, by e.g. Harremoes (Harremoës 1983; Lumbers 1983). These are all based on the treatment of soluble substrates, which is a limitation.

The major design parameter is organic loading per unit surface area of the rotating medium, usually expressed as g BOD/m^2 day. The retention time of wastewater in the biozone is also considered important.

Manufacturers size their units using design/performance curves, usually effluent quality against load, or sometimes effluent quality against hydraulic load at different wastewater strengths, Figure D1 gives two examples.

Various national bodies have produced prescriptive rules for sizing RBCs. BS 6297:1983 (British Standards Institution 1983) recommends the organic loading should be less than 5 g BOD/m^2 day into the biozone, or 7.5 g BOD/m^2 day of crude sewage into the primary settlement zone (based on the reasonable assumption that one third of the BOD is removed by primary settlement) to give an effluent quality of 20:30 mg/l BOD:SS. It is not stated whether this is a 95 percentile. Manufacturers may suggest other loadings provided sufficient technical support is given. The code of practice adds that 'where quality standards are critical, additional tertiary treatment should be provided'.

Table D1 shows loadings recommended by the German professional body ATV (Sewage Engineering Association 1983), to give a 20:30 effluent as an 80 percentile. The addition of flow balancing can allow the load to be increased up to 12 g BOD/m^2 day. This table specifically applies to RBCs sized for population equivalents greater than 500. For smaller sizes, which covers most of existing UK installations, the load should never exceed 8 g BOD/m^2 day. If the UK recommendations are for a 95 percentile effluent quality, these two guidelines are reasonably compatible.

Table D1 - German Regulations for design loading rates for RBC's

Treatment	Number of shafts in series	Overall organic loading (g BOD m².d)
Carbonaceous (BOD) removal	≥2	8
no nitrification	≥3	10
Carbonaceous removal with	≥3	4
nitrification	≥4	5

Scheible and Novak (1980), by conducting COD balances across a biozone, estimated the maximum oxygen transfer rate in an RBC at 7.3 g O_2/m^2 day. Since some BOD is removed by bioflocculation this might allow good BOD reductions at loadings somewhat higher than this figure. Since the nitrogenous oxygen demand of many domestic wastewaters is roughly equivalent to the BOD, and since in the more oxygen rich environment necessary for nitrification i) the driving force for oxygen transfer must reduce and ii) the ratio of biooxidation to bioflocculation will increase, it would follow that high rates of nitrification would not be expected at BOD loadings greater than 3-4 g BOD/m^2 day. This is now in the range of loadings that most manufacturers quote for RBCs designed for simultaneous BOD reduction and nitrification.

D3 PREDICTED AND ACTUAL PERFORMANCE

Because RBCs are used almost exclusively at small works, which always show extreme variability of performance, both in time and against design parameters, it is very difficult to give generalised performance figures. In any survey of RBC performance, the actual load is often the most uncertain figure. Published data from Wessex Water and North West Water (Performance of Packaged Sewage Treatment Plant 1986) show that of a number of RBCs loaded at from 2-8 g BOD/m^2 day, just over 50% produced an average effluent BOD of under 10 mg/l, which should meet a 20:30 95 percentile consent. A number of the remainder gave average effluent BODs of 10-20 mg/l while a few performed even less well. Within this range, performance did not correlate at all with BOD loading. At the time of this work, ammonia consents on small works were very rare so no data on nitrification were presented. In later work by WRc (unpublished) only 25% of a larger number of sites met 20:30 as a 95 percentile, though most sites had a more relaxed consent, which was met in about 50% of cases. About 25% of sites produced effluent ammonia concentrations less than 10 mg/l as a 95 percentile and 50% of sites produced effluent ammonia concentrations less than 10 mg/l as an average.

This lack of correlation between effluent quality and BOD loading is clearly predicted by the manufacturers own design curves. Figure D1 shows a variation in effluent soluble BOD from 3 to 4 mg/l as load moves from 4 to 10 g BOD/m^2 day (approximately 0.4 to 1.0 gal/ft^2 day) and Figure D2 shows effluent BOD quality varying from 9 to 12 mg/l over a similar loading range.

While the RBC is quite capable of producing a 20:30 95 percentile effluent, to do so reliably will require the addition of tertiary solids removal. It appears wrong to concentrate on BOD loading as the sole design parameter.

Secondary solids removal is of critical importance in all biological treatment processes, but several workers have highlighted this as a particular problem area for RBCs, Lumbers (1983), Tanaka, Oshima and Rittman (1987). Secondary solids are not as well stabilised as the humus solids from biological filtration, nor flocculated like in the activated sludge process. The mechanical forces to which RBC biomass is subject may encourage it to break up. Tanaka says that 30% of RBC secondary solids settle at less than 0.4 m/h.

D4 OPERATING CONSTRAINTS

D4.1 Effect of flow variability

At the small works where RBCs are generally sited, flow variations, both diurnal and wet weather, are often extreme. Many design schemes recommend a minimum retention time in the biozone of 1 h. High flow rates compound the problem of poor settleability of secondary solids. Flow balancing is to be recommended for RBCs working on flows from small populations especially when the flow is pumped.

D4.2 Denitrification

In common with other biological treatment systems, the nitrogen gas given off from denitrification can cause the secondary solids to float, leading to poor effluent quality. There is a common belief that RBCs are more prone to this than other biological treatment systems.

D4.3 Design

In attempting to produce fully-enclosed packages comprising primary and secondary settlement as well as biological treatment, many designs depart significantly from what would be considered good practice for separate units, particularly with respect to the settlement zones. How much this may effect performance is unclear but certainly many designs are very difficult to de-sludge effectively.

D4.4 Mechanical reliability

There is an undoubted questionmark against the mechanical reliability of RBCs. The mass of biomass and associated water in 1 m^3 of RBC media can approach 0.5 tonne. This may not always be distributed evenly around the RBC rotor. This leads to heavy stresses on motors, gear-boxes, bearings, axles, support struts and even the medium itself, all of which have been known to fail. Though there is little UK data in the public domain on this, Weston (1985) relates the position in the USA, and while the USA does have some much larger RBC installations the situation is undoubtedly very similar in the UK.

There seems no intrinsic reason why these problems cannot be overcome by good design, but perhaps economic pressures are preventing manufacturers from producing perfect solutions.

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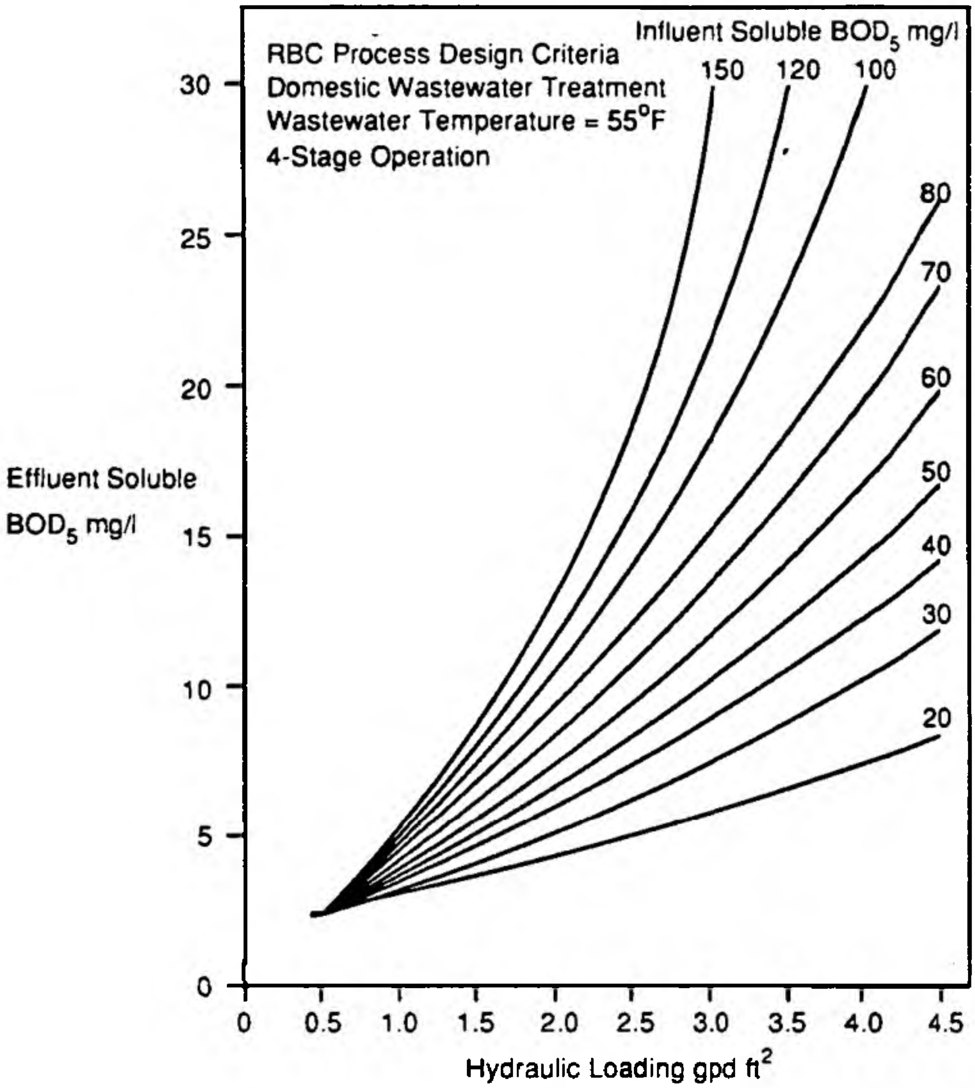

Figure D1 - Manufacturers design curves (Benjes, 1990)

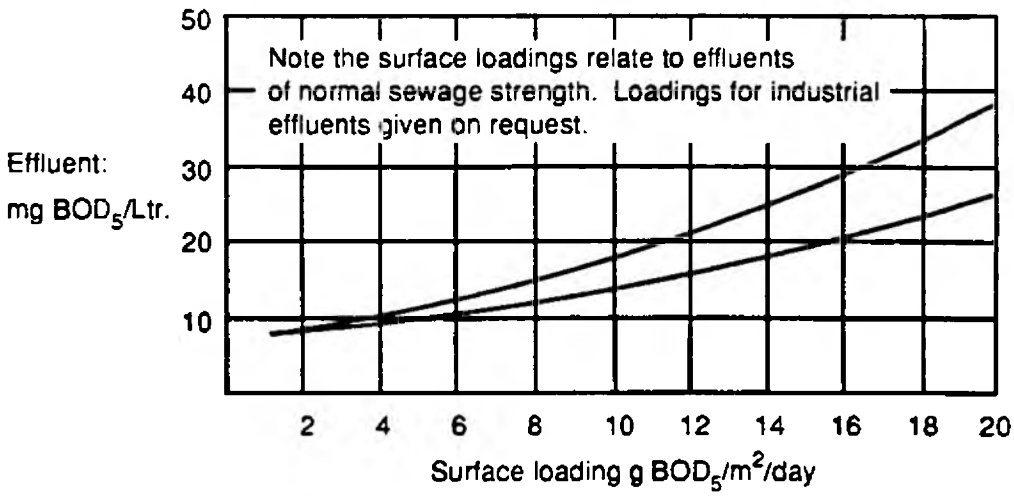

Figure D2 - Manufacturers design curves (UK manufacturer)

APPENDIX E - TERTIARY NITRIFYING FILTER

E1 PROCESS DESCRIPTION

Tertiary nitrifying filters are used to oxidise the ammoniacal-nitrogen in effluent from activated sludge and from biological filtration plant. Effluent trickles over nitrifying bacteria attached to the surface of irregularly-shaped medium which constitute the main part of the filter. The spaces between the medium allow natural ventilation by air which contains the oxygen used in nitrification. Given the lack of competition from heterotrophic carbonaceous bacteria, such filters allow the autotrophic nitrifying bacteria to grow rapidly enhancing nitrification performance over that of conventional biological filtration plant.

Typically nitrifying filters are about 3 m deep and contain granite or slag medium of about 28 mm size. The effluent is dispersed over the top of the bed through either a rotating distributor or a fixed distributor comprising a row of parallel pipes fitted with nozzles. The treated effluent drains through the elements of medium to a sloping floor, where drain tiles provide channels which allow unobstructed flow of treated effluent to an outlet.

E2 THEORY

About 4.3 g of oxygen are required to oxidise biochemically 1 g of ammoniacal-nitrogen to nitrate (Gujer and Boller (1986). The rate of nitrification depends on the area of biofilm available and hence the wetted surface area of the medium. Since the area of wetted surface increases with higher surface hydraulic loadings, the scientific expression commonly used to characterise filter performance is the ratio of the applied volumetric loading to the plan area of the filter, termed irrigation velocity.

Given the high oxygen requirement for nitrification, nitrifying filters are considered to be limited by oxygen dissolving in the effluent stream as well as influent ammonia. The reaction products, which comprise nitrate and carbon dioxide, diffuse from the biofilm into the effluent stream and are considered not to limit the nitrification rate.

Figure E1 presents the relationship between the instantaneous oxidation rate, ammoniacal-nitrogen concentration and irrigation velocity at different levels within a filter. At the top of a filter where the influent ammoniacal-nitrogen concentration is about 10 to 20 mg/l, nitrification is oxygen limited. The rate of nitrification is independent of the concentration of ammonia because the concentration of dissolved ammonia required for nitrification greatly exceeds the available dissolved oxygen. Further down the filter where ammoniacal-nitrogen concentrations of about 5 mg/l occur, nitrification starts to become ammonia-limited and the instantaneous oxidation rate begins to fall. At the bottom of the filter, nitrification is completely ammonia-limited and nitrification rates fall markedly. Therefore filter volume has to be increased significantly to achieve low average concentrations of ammonia such as 1 or 2 mg/l.

Figure E1 also indicates that nitrification rates are increased by increasing the irrigation velocity. This arises because the wetted area of medium increases with higher velocities. Accordingly recirculation of filter effluent to increase the irrigation velocity can improve performance.

E3 PREDICTED AND ACTUAL PERFORMANCE

As described above the nitrification rates in biological filters are related to the surface area of the medium and irrigation velocity. Therefore the most suitable loading criterion would be in terms of a combination of such variables. Given that modern practice is to construct such filters about 3 m deep and use 28 mm medium, the loading criteria can be simplified and volumetric loading adopted. Clearly if medium larger than 28 mm is used, performance will deteriorate.

A mathematical model incorporating the theory presented above has been used to predict the performance of a 3 m deep filter filled with 28 mm medium and treating effluent with an ammoniacal-nitrogen concentration of about 15 to 25 mg/l. Simulations have been performed for irrigation velocities ranging between 10 and 50 m/d and operating temperatures pertaining to the yearly-average and to winter. The results of the simulations presented in Figure E2 relate effluent ammoniacal-nitrogen concentration to volumetric ammoniacal-nitrogen loading and include actual performance data available from general literature for full-scale plant operating for several years at Wanlip WRW (Upton and Cartwright 1984) and pilot plant at Davyhulme STW (Dolan and Horan 1988) operating over a period of 2 months. It highlights the following points:

- Given the complex relationship between effluent and influent ammoniacal-nitrogen concentration and irrigation velocity, the use of volumetric loading based on ammoniacal-nitrogen concentration gives a broad range of performances.

- Yearly-average effluent ammoniacal concentrations of about 1 mg/l can be achieved in practice.

- **The performance of the filters deteriorates significantly in winter owing to the lower effluent temperatures.**

- Water companies operate at volumetric ammoniacal-nitrogen loadings of about 50 to 60 gN/m3d to maximise the performance of nitrifying filters.

Experience within the industry suggests thst the ratio of the 95 percentile performance to the average performance varies from about 2.5 to 3.0. Use of the ratio indicates that **the 95 percentile performance of such filters is about 2 to 3 mg/l.** To achieve lower effluent ammonia levels, trials are needed to demonstrate the performance of such filters at lower loadings.

E4 OPERATING CONSTRAINTS

E4.1 Effect of influent BOD

The performance of a nitrifying filter can be impaired if the influent contains high BOD levels. It is probably caused by increases in the population of carbonaceous bacteria reducing both the space on the medium and the oxygen available for the growth of nitrifying bacteria. Impairment in performance can be limited by operating the upstream process optimally and maintaining influent BOD's to the filter below 30 mg/l.

E4.2 Alkalinity

Nitrification in biofilms consumes alkalinity and liberates carbon dioxide. If the influent has a high ammoniacal-nitrogen concentration which is above about 50 mg/l or insufficient alkalinity indicated by a pH which is below about 6, filter performance can be inhibited. Normally most effluents contain sufficient hardness for such inhibition not to be significant.

E4.3 Effect of diurnal load variations

A nitrifying filter fed at a constant rate operates optimally. Such a constant feed rate can be achieved by one of two methods. Either the flow of influent can be supplemented during low flows by recirculating filter effluent or balancing tanks can be installed upstream of the filters to allow the flow of influent to be supplemented during the daily periods of low flow.

E4.4 Effluent clarification

Since the nitrifying bacteria oxidise some of the carbonaceous BOD in the influent, nitrifying filters typically reduce the BOD by about 30%. The seasonal sloughing that occurs in conventional biological filters is not significant in nitrifying filters, also the sludge yield for nitrifying filters is much lower than for BOD removal processes. Accordingly clarification tanks are not normally installed after nitrifying filters. The preceeding activated sludge or biological filtration plant generate most of the biological solids produced at a works and hence the existing settlement tanks are adequate to meet the quality of solids required in the final effluent.

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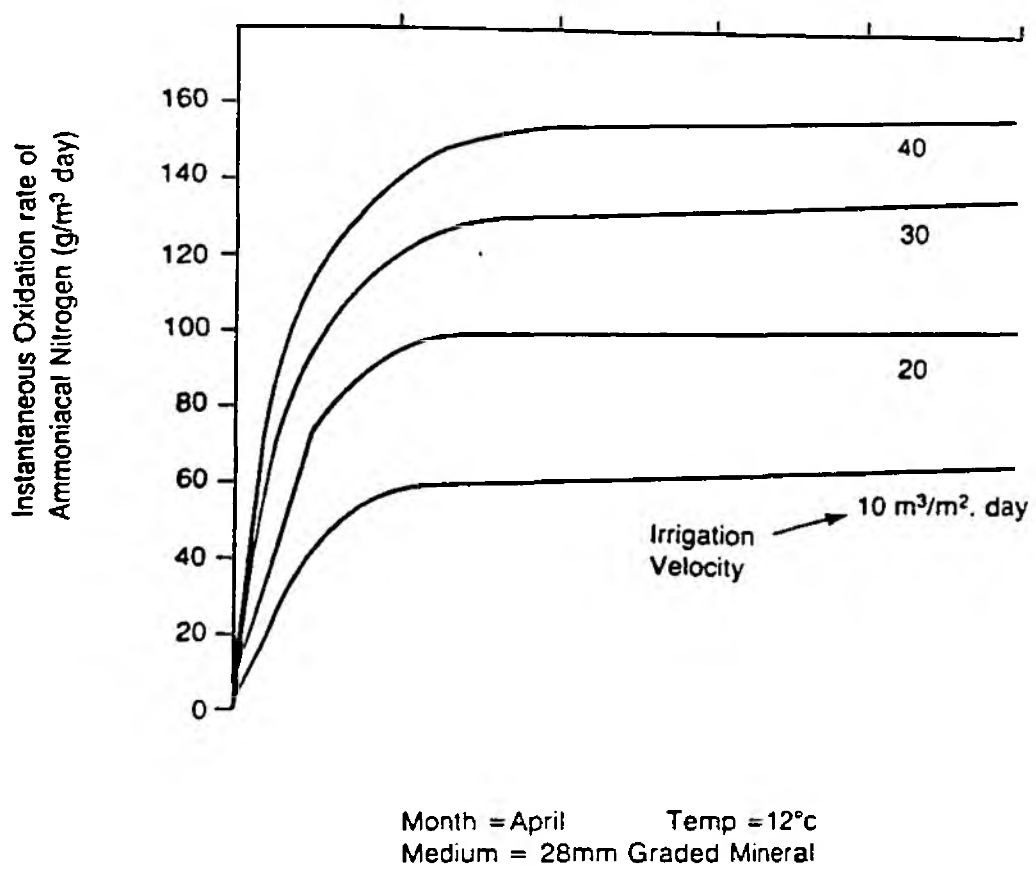

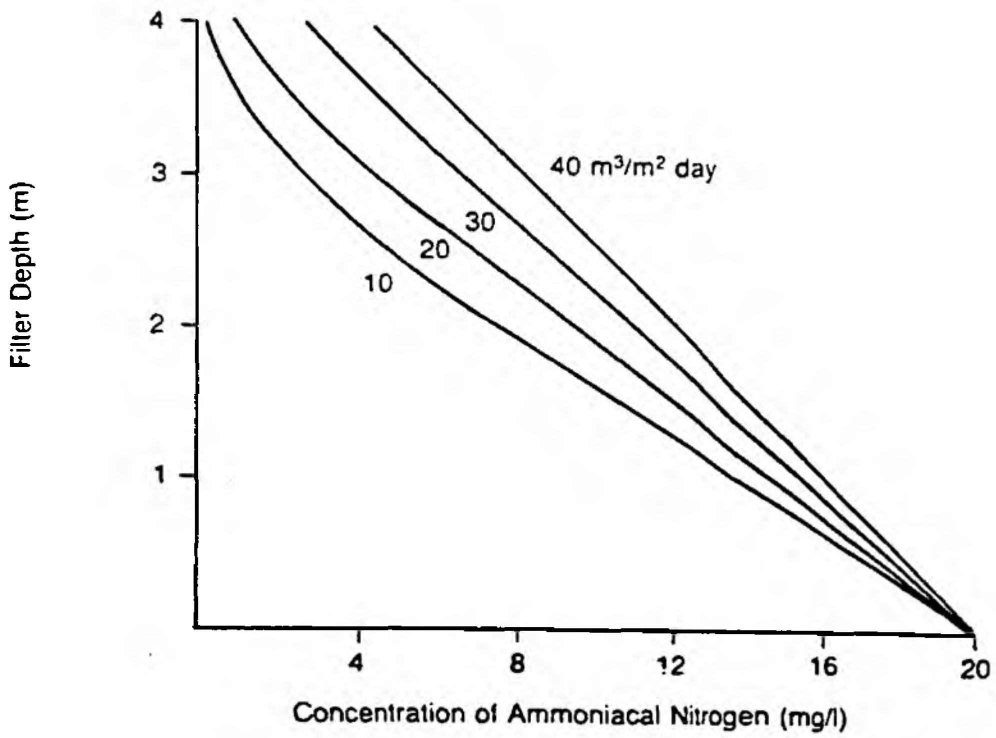

Figure E.1 - Relationship between nitrogen concentration, oxidation rate and filter depth

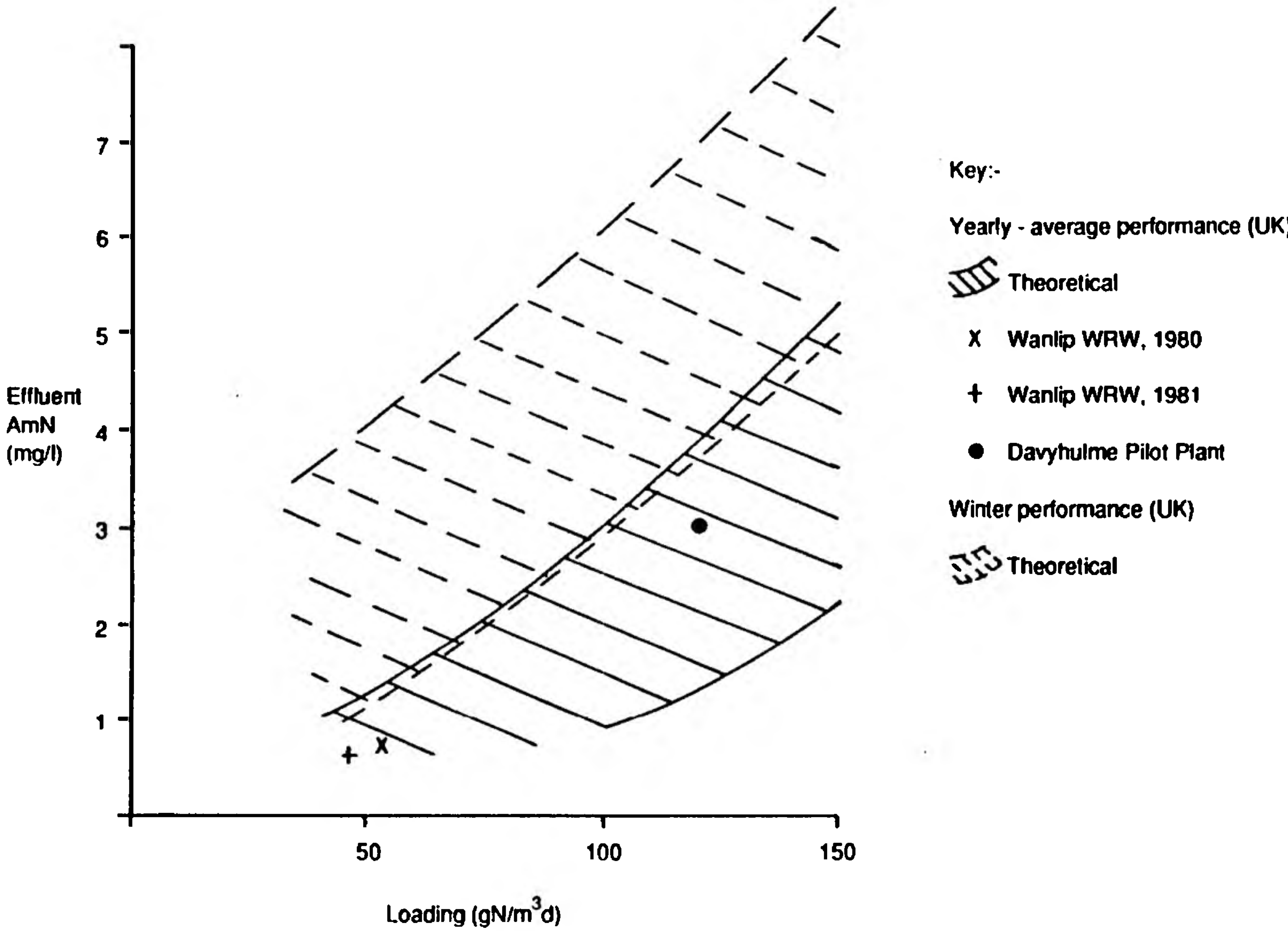

Figure E2 - Tertiary nitrifying filter: Effluent AmN versus AmN loading

APPENDIX F - TERTIARY SAND FILTERS

F1 PROCESS DESCRIPTION

Sand filters are used to treat the effluent from biological filtration and activated sludge plant. The effluent suspended solids treated are normally below 30 mg/l.

The equipment for tertiary treatment is described in the Manual of British Practice (Institute of Water and Environmental Management 1974). Slow sand filters are the earliest form. Such filters comprise a top layer of sand, about 0.15 to 0.7 m deep, containing grains with a particle diameter between 0.3 and 0.5 mm to strain solids from effluent and a bottom layer of gravel about 0.15 to 0.7 m deep, containing elements with a diameter between 10 and 50 mm to allow drainage. They operate at low irrigation velocities of 2 to 3 $m^3/m^2.d$ to allow the development of a biologically-active surface layer which degrades solids. Such filters do not require backwashing.

In view of the large area of land required for such filters, they are rarely used and have been superseded by high rate units. Such units comprise beds about 1 m deep filled with sand of about 1 to 2 mm diameter and they operate at irrigation velocities of over 100 m^3/m^2d. Accordingly the capital cost of such filters are significantly reduced over low-rate units but the sand needs to be cleaned frequently to remove the accumulation of solids that would otherwise block the filter.

There are two main types in use. Downward flow units treat influent entering the top of the filter and are backwashed daily using an air scour followed by a high flow of filter influent which enters the bottom of the unit and flows upwards. Upward flow units treat effluent entering the bottom and are backwashed daily using an air scour followed by a high flow of filter effluent which also enters from the bottom of the unit.

Multiple units are used to allow the flow to be continuously treated, while a filter is being backwashed. The minimum number of units installed is normally four. Accordingly to reduce the number of units required and minimise capital expenditure filters have been developed in which sand is continuously removed for cleaning and then returned.

The remainder of this appendix refers to high-rate filters only.

F2 THEORY

Solids removal occurs through (i) transport which brings the solids into the close vicinity of a element of medium and then (ii) attachment to the surface of the element. The mechanisms of solids transport and the underlying forces responsible for attachment are described elsewhere (Jago 1977).

Critchard, Fox and Green (1979) performed a comparison of an upward and downward flow filter using two 0.38 m steel tanks filled with sand of 1 mm effective size. The results presented in Figure F1 relate the performance of the two filters to irrigation velocity. Each filter was operated for a period of 14 days at each irrigation velocity and the influent and effluent suspended solids pertain to the mean value determined from 24 hour composite samples taken over the period. The results indicate that both filters achieved effluent suspended solids of about 6 mg/l, with an optimum irrigation velocity of about 100 m^3/m^2d for the downward flow unit and an optimium velocity of about 200 m^3/m^2d for the upward flow unit.

The explanation of the higher performance of upward flow units is as follows. Most filters contain a single size range and type of medium, which are graded through the filter as a result of backwashing with the smaller medium at the top and the larger medium at the bottom. In downward flow filters the solids have a tendency to collect as a surface layer at the top of the filter, while in upward flow filters the larger medium at the bottom allow the solids to be distributed through a greater depth of the filter. Accordingly upward flow filters have a greater capacity for solids than downward flow filters enabling them to accept higher loadings.

Given that units are designed on the maximum flow to treatment and that the ratio of the maximum flow to the dry weather flow is three, modern practice is to design downward flow units at an irrigation velocity of 250 m^3/m^2d and upward flow filters at an irrigation velocity of 400 m^3/m^2d.

About 4.3 g of oxygen are required to oxidise biochemically 1 g of ammoniacal-nitrogen. Given that the filters are not aerated and that the maximum level of dissolved oxygen in the influent is about 8 to 10 mg/l, the filters can at best only remove about 2 mg/l of ammoniacal-nitrogen. Hence such filters should not contribute significantly to ammonia removal.

F3 PREDICTED AND ACTUAL PERFORMANCE

The performance of a large number of upward flow and downward flow units are summarised in Tables F1 and F2. The results indicate that the performance of such filters exhibit some variation between works but generally achieve a mean effluent suspended solids level of about 6 mg/l treating flow from a biological filtration plant and levels as low as 4 mg/l for activated sludge plants.

Table F3 presents values of the ratio of the 95 percentile to mean value for works at Addison (USA) (Fitzpatrick and Pipes, Unpublished) and Letchworth (Truesdale and Birkbeck 1968). As expected the value of the ratio is about 2, which is the value commonly found in other processes treating a diurnally-varying flow. Accordingly for the typical loadings given above, sand filters should meet a 95 percentile value of 15 SS mg/l. When treating effluents from a nitrifying activated sludge plant, they should achieve a 95 percentile value of 10 mg/l.

F4 OPERATING CONSTRAINTS

The biological solids in sewage effluents are tenacious and if not removed from medium by backwashing can cause partial blockage of the filter impairing performance. Backwashing generates turbulent conditions in the filter by using a combination of air scour to loosen the solids followed by a high flow of effluent to flush the unattached solids from the filter. **Efficient backwash systems are therefore essential.**

Table F.1 - Performance of rapid downward-flow sand filters

LOCATION	NUMBER OF FILTERS	PLAN AREA OF FILTERS (m^2)	MEDIUM			LOADING ($m^3/m^2.d$)		PLANT SUPPLYING INFLUENT	SUSPENDED SOLIDS (mg/l)		BOD (mg/l)		REFS
			DEPTH (m)	TYPE	SIZE	Max.	Mean		IN	OUT	IN	OUT	
Coventry	6	101	0.61	Sand	0.6-1.0	-	120	Biological filter	21	8	18	7	F1
East Hyde, Luton	9	176	0.2 0.88	Gravel + Sand	3-13 0.9-1.7	-	86	Activated sludge + biological filter	30	4	18	6	F4
Rye Meads	-	104	-	-	-	272	200	Activated sludge	10	3.8	-	-	F9
Crawley	-	229	-	-	-	250	100	-	19	6.0	-	-	F9
Swindon	-	354	-	-	-	190	90	Biological filter	22	6.2	-	-	F9
Milton Keynes	-	58	1.0	Sand	1.2-2.4	-	52	Activated sludge	18	7.0	-	-	F4
Wellingboro	1	0.41	1.0	Sand	1-2	-	200	Biological filter	16	4.5	-	-	F5
"	1	0.29	1.22	Anthracite		-	600	Biological filter	21	7.0	-	-	F5
West Germany	-	-	0.85	Sand	1.6-1.8	-	240	-	27	4.8	-	-	F6
"	-	-	1.2	Sand + Anthracite		-	240	-	27	3.0	-	-	F6
"	-	-	1.2	"		-	86.4	-	13.7	4.5	-	-	F6
"	-	-	0.25	Sand	0.45	-	288	-	10.7	3.1	-	-	F8

Table F.2 - Performance of rapid upward flow sand filters

LOCATION	NUMBER OF FILTERS	PLAN AREA OF FILTERS (m^2)	MEDIUM			LOADING ($m^3/m^2.d$)		PLANT SUPPLYING INFLUENT	SUSPENDED SOLIDS (mg/l)		BOD (mg/l)		REFS
			DEPTH (m)	TYPE	SIZE	Max.	Mean		IN	OUT	IN	OUT	
Easthampstead	4	117	2.35	Sand + Gravel + Gravel + Gravel	1-2 2-3 8-12 40-50	-	62.3	-	32	3	30	9	F1
East Hyde, Luton	12	234	1.5 0.6	Sand + Gravel	0.9-1.7 6-50	-	135	Activated sludge + biological filter	30	4	18	6	F1
Chalton, Luton	8	220	1.5 0.45	Sand + Gravel	1-2 2-50	-	95	Biological filter	19	6.5	-	-	F4
Aldershot	1	0.46	1.5 0.65	Sand + Gravel	1-2 2.8-56	-	107 250 406	Biological filter	25 32 30	5.8 8.4 11.2	- - -	- - -	F3
Camberley	-	117	-	-	-	410	170	-	22	6.0	-	-	F9
Sandhurst	-	116	-	-	-	380	100	-	25	5.5	-	-	F9
Blackbirds	-	551	-	-	-	220p	90	-	36	4.3	-	-	F9

Table F.3 - 95 percentile performances of filters

LOCATION	NUMBER OF FILTERS	PLAN AREA OF FILTERS (m^2)	MEDIUM			MEAN LOADING (m^3/m^2.d)	SUSPENDED SOLIDS (mg/l)				BOD (mg/l)			
			DEPTH (m)	TYPE	SIZE		MEAN INFL.	MEAN EFFL.	95%ILE EFFL.	95%ILE/ MEAN EFFL.	MEAN INFL.	MEAN EFFL.	95%ILE EFFL.	95%ILE/ MEAN EFFL.
Addison North Plant (USA) (NOTE A)	3	10.5	0.53 0.61	Anthracite Sand	2.7 1.4	220	33.9	5.1	14	2.7	43.9	6.6	14	2.1
Addison South Plant (USA) (NOTE B)	3	10.5	0.23 0.56	Anthracite Sand	1.7 0.7	120	41.8	9.7	25	2.6	37.5	9.1	16	1.8
	4	10.5	0.38 0.58	Anthracite Sand	2.7 1.4	120								
Letchworth STW (NOTE C)	1	0.45	1.5	Sand	1-2	200	-	-	-	-	21.5	5.5	10	1.8
						286	-	-	-	-	15.8	6.7	11	1.7
						416	-	-	-	-	14.2	8.8	13.5	1.5

NOTE:

A The North plant treats effluent from a contact stabilisation process.

B The South plant treats effluent from biological filter and activated sludge plants operated in parallel.

C It comprises an upward flow filter operated at a constant feed rate treating effluent from an activated sludge plant.

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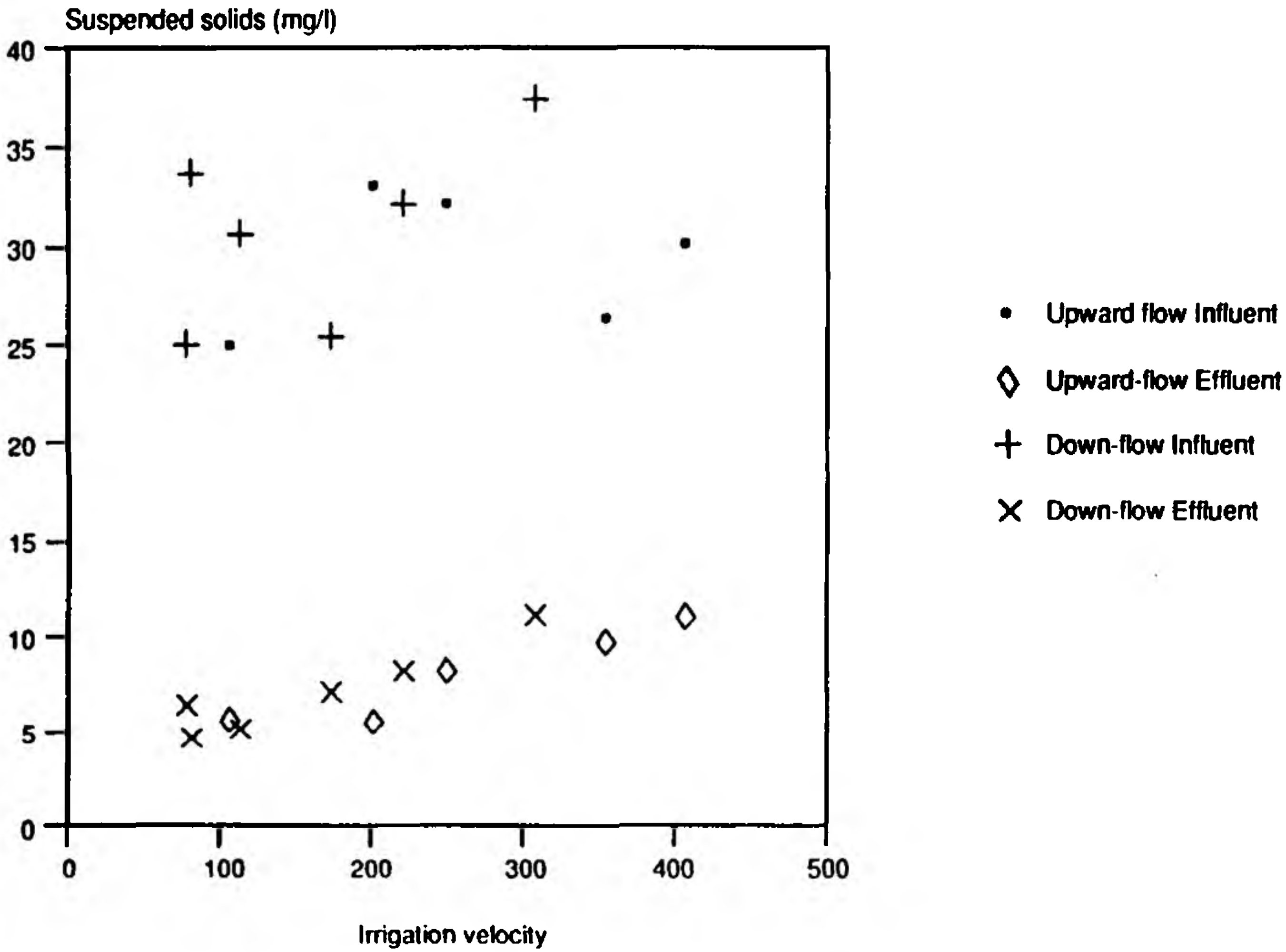

Figure F1 - Tertiary sand filter: Effect of irrigation velocity on perfomance

APPENDIX G - CHEMICAL DOSING TO REMOVE PHOSPHORUS

G1 PROCESS DESCRIPTION

Phosphorus is precipitated from wastewater by dosing chemicals into or after the biological treatment process. The chemicals are added using a dosing pump, regulated by a control system. The chemical should be added to the wastewater at a point in the system that is well mixed to encourage precipitation. The point of addition is dependant upon type of secondary treatment process that is being employed.

G1.1 Activated sludge

The chemical precipitant is usually added either to, or just prior to, the aeration tank and precipitate is removed with the sludge in the final clarifier. This process is known as simultaneous precipitation. Aeration causes some chemicals to be oxidised (see Section G4.2). The chemical can also be added prior to primary treatment, or post secondary treatment. The resulting precipitate is removed either with the primary sludge or by an extra on-line filter.

G1.2 Biological filtration

The chemical precipitant can be added prior to either the biological filters or the humus tanks. Because of operational difficulties the latter process is usually employed.

G2 DOSING METHODS

When dosing a chemical precipitant to wastewater, a control system is needed to accurately inject the chemical and reduce, if not eliminate, any excess chemical entering the system. Three main methods of dosing are used to remove phosphorus.

G2.1 Chemical dosing system

No control exists in this system and the chemical is dosed at a set flowrate using a plunger-type pump. This method usually causes a large excess of chemical to be added.

G2.2 Timer controlled dosing

In this method the pumps are controlled on timers such that they operate at times of peak flow and switch off when the expected flow is low. This system reduces but does not eliminate wasteful dosing.

G2.3 Redox controlled dosing

When chemicals are added to wastewater the redox potential rises. Control systems can be designed to reduce dosage whenever the redox increases. Such systems eliminate excess dosing.

G3 THEORY

Phosphorus is present in wastewaters in three forms: orthophospate ion, polyphosphates and organic phosphorus compounds. Both polyphosphates and organic phosphorus compounds decompose during secondary biological treatment to form orthophosphate ions. From the viewpoint of chemical precipitation, this is beneficial since the orthophosphate form is the easiest to precipitate. There are a number of forms of orthophosphate in equilibrium, with the predominant form and they change with pH. The standard form of orthophosphate in wastewater is PO_4^{3-}.

When a metal salt is added to the wastewater, the precipitation of phosphorus can be expressed as follows:

$$M^{3+} + PO_4^{3-} \rightarrow MPO_4 \downarrow$$

The MPO_4 compound is removed with biological sludge in a clarifier or humus tank.

G4 RAW MATERIALS

The materials which are most suitable for precipitation of phosphorus are the ionic forms of aluminium, iron and calcium.

G4.1 Aluminium compounds as precipitants

Aluminium sulphate (Alum) and sodium aluminate are the two aluminium compounds which are suitable for phosphorus precipitation. Both compounds are available in solution or in dry form. Alum can be stored indefinitely in solution but is expensive to transport since it contains up to 50% v/v water. Sodium aluminate can be stored for three months in liquid form or six months in dry form. Both forms are corrosive.

The optimum pH for the precipitation reaction is in the range 5.5 - 6.5. When Alum is added the reaction tends to lower the pH of the wastewater. It will not generally reduce the pH below 6.0 but if this occurs addition of an alkaline substance is required. Sodium aluminate raises pH, and should only be used on acidic wastewaters.

Neither of these compounds are used in UK at present.

G4.2 Iron compounds as precipitants

Ferric (Fe^{3+}) and Ferrous (Fe^{2+}) salts are used to precipitate phosphorus from solution. The forms of the salts are usually chlorides and sulphates. Chloride salts are extremely corrosive and must be stored in a heated environment. Sulphate salts are not as corrosive and may be stored in a cool dry area in dry form. For ferric salts the optimum pH range is 4.5 to 5.0. Since achieving such a low pH value would entail addition of acid, which is expensive, the reaction usually takes place in an environment of pH 5.0 - 6.0. For ferrous salts the optimum pH is approximately 8, with good removal occurring in the range 7 to 8. **Iron compounds are used as precipitants at about 20 sewage works in the UK.**

Note that when a ferrous salt is added to an activated sludge plant, it is oxidised to a ferric salt in the aeration tanks. Such an addition gives low values of phosphorus in the effluent, whilst still allowing sufficient phosphorus to be available for the micro-organisms in the activated sludge process.

G5 EFFLUENT QUALITY

The removal of phosphorus from wastewaters by precipitation can reduce average effluent phosphorus values to as low as 0.5 mg-P/l. To attain such low levels, the quantity of precipitant needed greatly increases (See Table G1).

Table G1 - Chemical precipitant

Averave effluent phosphorus level (mg-P/l)	Process option	Precipitant	Mole ratio
2-3	Simultaneous Precipitation	Fe(II);Al(III)	0.8
	Pre-ppt	Al(III)	1.0
1-2	Simultaneous ppt	Fe(II);Al(III)	1.0
	Pre-ppt	Al(III)	1.0
	Post-ppt	Fe(II)	1.0
0.5-1	Simultaneous ppt	Fe(II);Al(III)	1.5
	Pre ppt	Al(III)	2.0
	Post ppt	Fe(II)	1.5

To achieve effluent phosphorus levels below 0.5 mg-P/l some form of tertiary treatment is usually necessary to remove the fine solids which may contain some of the precipitated phosphate. This usually entails extra chemical dosing which will remove the phosphorus after secondary settlement. An on-line filter is normally needed to remove the extra solids created by precipitation.

G6 OPERATING CONSTRAINTS

G6.1 Ponding of filters

If the precipitant is dosed prior to the filter bed so that complete removal of phosphorus takes place, a large quantity of insoluble precipitate is transferred onto the medium of the filter. This precipitate can build up in the interstices and cause ponding where the accumulation of solids clog the filter leading to poor drainage and ventilation. The end result of ponding is reduced efficiency or complete breakdown of the process.

G6.2 pH effects

If the precipitant is added before the secondary treatment process the pH of the wastewater should not be lowered to a point where the biomass in secondary treatment cannot function effectively.

G6.3 Minimum P-concentration

It is commonly accepted that a minimum concentration of phosphorus in relation to BOD of 1:100 is required in order to ensure that the nutrients are not limiting. Otherwise if all the phosphorus is removed prior to biological treatment insufficient nutrients will be available to encourage growth impairing performance.

G6.4 Pollutant addition

By dosing chemicals to wastewater to remove phosphorus a foreign substance is being introduced to the watercourse. If the dosing of this chemical is not strictly controlled, significant quantities of Iron or Aluminium salts may pass into the watercourse causing pollution.