

# **IN-SEWER TREATMENT OF DOMESTIC WASTEWATER**

by

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Thesis submitted for the degree of

Doctor of Philosophy

April 1996



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Dedicated  
to  
my  
parents

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## ACKNOWLEDGEMENTS

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This work would not have been completed without support from many quarters. First and foremost, I owe a great debt of gratitude to my supervisors Prof. G. K. Anderson and Mr. D. J. Elliott for their invaluable help and guidance given during the course of this study. I am also indebted to Prof. M. B. Pescod, Mrs L. M. Evison, Dr. Tom Curtis, Dr. A. V. Metcalf and Prof. G. P. Noone, for their help at various stages of this study. The assistance of administrative and technical staff in the Department of Civil Engineering, including Pat Johnston, Amanda Allan, John Hamilton, Johan Baugh and Lez Hepple, is gratefully acknowledged.

Dr. Staurt McGulshan of BOC, Mr. Merve Ogston of CIG Australia, Dr. Y. S. Cao of Delft, The Netherlands, Dr. Kama Raunkjaer of Aalborg University, Denmark, Dr. Adem Ozer of Dokuz Eylul University, Turkey, Dr G. Zeeman of Agricultural University Wageningen, The Netherlands, and Mr Bryan Hancock of Irvine Ranch water District, California, provided useful literature concerning various aspects of this research, and I would like to extend my sincere thanks to all of them for their valuable assistance.

I am grateful to Tameez Ahmad, Mohesni Bandpi, Raja Rafhat Iqbal, Amer Islam Sheikh, M. Shoaib Razzaq, Dr. Khawaja Farooq Ahmad, Md. Mehbub Hasan, Ali Mahdavi, Dr. Muna Hindiye, Zaini Ujang and Dr Noel Craine for their assistance, friendship and encouragement during my stay at Newcastle.

My grateful thanks are due to the Secretary General Water Authority of Jordan, for allowing me to use research facilities at the Authority's Central Laboratories in Amman. Special thanks are due to Dr. Raja Gedeon, Abdual Wahb Matr, Nawal Sunna, Suzan-al-Kilani, Omar Khitab, Dr. Muwafiq Saqqar, and all the staff at Wadi Seer Laboratories in Amman.

I wish to thank the Ministry of Science of Technology, Government of Pakistan, for their financial support and the Department of Communication & Works, Government of NWFP, Pakistan, for granting study leave. Thanks are also due to the Committee of Vice-

Chancellors and Principals of the Universities of the UK for an ORS award, and the University of Newcastle upon Tyne for the TULIP award and for supporting research visits in the UK and abroad.

Last but not least, I would like to express my deep appreciation to my mother, sisters, and brothers Zaffar Iqbal Malik and Mustafa Kamal Malik. This work would not have been possible without their undiminished love and countless sacrifices.



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## ABSTRACT

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Urban sewerage systems, which are normally used for the transport of wastewater from its origin to a wastewater treatment plant (WWTP), could be used as a treatment facility because (i) they contain heterotrophic bacteria capable of oxidising organic matter both suspended within the body of the flowing wastewater and attached to the surface of the wetted perimeter, and (ii) they provide retention times which are often comparable to those in a conventional activated sludge aeration tank and which, in some cases, may be equal to the hydraulic retention time in a WWTP. Using sewers as a treatment facility could be an economical method of alleviating the load on an existing WWTP or reducing the size of the proposed WWTP.

The current study was undertaken to investigate the feasibility of using urban sewerage systems as suspended growth biological reactors for the treatment of domestic wastewater. The flow in a linear gravity sewer was simulated using a batch reactor fed with raw domestic wastewater. A comparison of simulated aerobic and anaerobic gravity transport indicated that aerobic treatment would be the most favoured method of in-sewer biological treatment. The soluble COD (SCOD) removal efficiencies over a retention period of 8 hours averaged 36 and 6% under aerobic and anaerobic conditions, respectively, at an average temperature of 22°C. The corresponding total COD removal averaged 8 and 11%, respectively. When the effluent samples, taken from the batch reactors after a retention period of 6 hours, were settled in a bench-scale settling column for one hour, the average suspended solids removal under aerobic conditions was 29% greater than those under anaerobic conditions.

Under aerobic conditions, the removal of soluble organic matter during simulated gravity transport was found to be strongly influenced by the strength of the incoming wastewater. To investigate the effect of wastewater influent soluble COD ( $\text{SCOD}_0$ ) and influent suspended solids ( $\text{SS}_0$ ) on in-sewer aerobic treatment, 27 individual wastewaters collected from the inlets to three wastewater treatment plants were subjected to batch tests at  $20^\circ\text{C}$ . The SCOD over a retention period of 8 hours at  $20^\circ\text{C}$  averaged 48, 40 and 61% for wastewaters having low  $\text{SCOD}_0$  and low  $\text{SS}_0$ , high  $\text{SCOD}_0$  and low  $\text{SS}_0$ , and high  $\text{SCOD}_0$  and high  $\text{SS}_0$ , respectively. The corresponding soluble  $\text{BOD}_5$  removal efficiencies averaged 64, 59, and 81%. A statistical analysis of soluble COD data revealed that, over a retention period of one to three hours, the soluble COD removal is only significantly influenced by  $\text{SS}_0$ . At higher retention periods, the soluble COD removal was found to be significantly affected by both  $\text{SCOD}_0$  and  $\text{SS}_0$ . Soluble COD removal was found to follow first-order kinetics with respect to time. The oxygen uptake rate of the individual wastewaters varied widely and did not appear to show any clear relation with the  $\text{SCOD}_0$  or  $\text{SS}_0$ .

An increase in the suspended biomass of the wastewater by the addition of activated sludge, at a concentration as low as 100 mg VSS/l, at the inlet of the simulated aerobic gravity sewer resulted in a significant increase in the removal of soluble organic matter. The soluble COD removal in the seeded wastewater was found to increase almost linearly with the increase in seed concentration in the range of 100-1000 mg/l. The effect of seed concentration on soluble COD removal however, appeared to diminish with the increase in retention time. SCOD removal in the seeded wastewater appeared to follow second-order kinetics with respect to time. At an initial seed concentration of 100-1000 mg/l, the batch reactor's effluent after a retention period of 6 hours showed satisfactory settling

characteristics. The oxygen uptake rate of the seeded wastewater did not show any specific trend over time at seed concentrations of 100 and 250 mg/l, while at higher seed concentrations it was similar to that observed in a typical plug flow activated sludge aeration tank.

The results of the case study in which the wastewater collected from the inlet of the Greater Amman Siphon (GAS) was maintained aerobic in a batch reactor, showed that by maintaining aerobic conditions in the GAS, average SCOD and soluble BOD<sub>5</sub> removal efficiencies of 60 and 78%, respectively, could be achieved over 8 hours at an average temperature of 25°C. The average oxygen demand of the wastewater was estimated to be 30 mg/l.h.

The result of the current study suggest a strong possibility of using urban sewerage systems as an aerobic biological reactor for the removal of soluble organic matter during transit.

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## ABBREVIATIONS AND NOMENCLATURE

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°C	Degrees Celsius
atm	Atmospheres (pressure)
ATU	Allylthiourea
B	Biomass or seed concentration
b	$\{1+B/100\}$
BOC	British Oxygen Company
BOUR	Oxygen uptake rate of the biofilm or slime layer
CFSTR	Continuous flow stirred tank reactor
CIG	Commonwealth Industrial Gases (Australia)
d	Day
DO	Dissolved oxygen
h	Hour
ISAT	In-sewer aerobic treatment
kg	Kilograms
km	Kilometers
l	Litres
m	Meter
mg	Milligrams
min	Minute
mwg	Meters water gauge
n	Number of replicate experiments
O <sub>2</sub>	Oxygen
p	Minimum level of significance at which an hypothesis can be rejected
RAS	Returned activated sludge
rpm	Revolutions per minute
RSCOD	SCOD removal in mg/l
SBOD <sub>5</sub>	Five days soluble biochemical oxygen demand
SCOD	Soluble chemical oxygen demand

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SCOD <sub>0</sub>	Influent SCOD
SD	Standard deviation
sec	Second
SOUR	Oxygen uptake rate of suspended biomass
SS	Suspended solids
SS <sub>0</sub>	Influent SS
SSVI	Stirred sludge volume index
SVI	Sludge volume index
t	Retention time
VSS	Volatile suspended solids
WAJ	Water Authority of Jordan
WWTP	Wastewater treatment plant

## CHAPTER ONE

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### INTRODUCTION

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The possibility of using the sewerage system to achieve treatment of a wastewater during its transit would be an attractive option in any wastewater management strategy. In general, urban sewerage systems are designed purely to transport wastewater from its origin to wastewater treatment works. If it was possible, however, to treat the wastewater on route to its destination, then the size and cost of any required treatment works would be greatly reduced.

The main objectives of any domestic wastewater treatment plant are, in essence, the same. In the United Kingdom, these objectives are embodied by the EC Urban Wastewater Treatment Directive. To protect the environment, the function of any wastewater treatment plant is to reduce both the organic strength of the wastewater and the quantity of suspended solids in the effluent, to levels prescribed by the Directive. If these prime objectives and the methods of achieving them are considered, it begins to become clear that the prevailing conditions in a sewer may be well suited to providing at least partial treatment of the wastewater.

In general, a typical sewer acts as a plug flow reactor (Hemmings *et al.*, 1983) with biomass both suspended within the body of the flowing wastewater, and attached on the wetted perimeter. The retention time in the sewer is dictated by the sewer length and the flow rate of the wastewater. In the case of a very long sewer, the wastewater transit time may be in excess of the retention time at any downstream treatment works. Additionally,

the natural flow in the sewers provides ideal conditions of mixing for bacterial growth and flocculation. Without engineering the sewer in any special way, it is clear that some of the basic requirements of a biological treatment process are satisfied by simple flow in a sewer. By actively promoting other conditions in the sewer, the potential for in-sewer biological treatment becomes greatly increased.

Of the various methods of biological treatment, it is likely that aerobic biological treatment will prove to be the most feasible because of the fact that in-sewer air and oxygen injection is widely practised for sulphide control, and the potential odour and corrosion problems associated with anaerobic conditions may rule out treatment by anaerobic methods.

Several alternative methods of aerobic treatment exist that may be applicable to in-sewer situations. First, the indigenous biological population, present in the wastewater and in the naturally occurring wall slimes, will contribute to the natural degradation of organic matter. Given suitable conditions and sufficient time, a significant level of treatment may be feasible without introducing additional microorganisms.

If higher rate methods are desired, however, it is possible that either suspended growth or fixed film processes may be applicable. Obviously, these methods require some 'engineering' of the sewer environment to be feasible. For example, suspended growth systems require a suitable source of seeding culture and fixed film methods require an extended surface area.

A small treatment works located near the head of the sewer leading to a regional treatment works could serve as a feasible source of seeding (Greenfield, 1980). The excess activated sludge instead of being wasted to the digester could be added to the head of the sewer if an upstream treatment plant is not readily available, a portion of the main flow could be drawn off and treated. Alternately, it may be feasible to recycle the sludge from a downstream plant via an aerated pressure main and then use a downhill gravity sewer as the mixed liquor tank (Pomeroy and Lofy, 1977).

The surface area in a pressure main could be increased by using a number of small diameter pipes instead of a large pipe or by filling one large pipe with a number of small pipes (Newcombe *et al.*, 1979). In gravity sewers, the surface area could be increased by lining the lower half of the sewer with plastic sheet having vertical vanes projecting into the stream, aligned parallel to the axes of the pipe (Pomeroy and Lofy, 1977).

The suspended growth in-sewer treatment system appears to be more practicable considering the fact that increasing the surface area would result in increased friction losses in pressure mains and higher capital costs for both pressure mains and gravity sewers (Pomeroy and Lofy, 1977; Newcombe *et al.*, 1979).

The advantages of in-sewer treatment methods are numerous. As stated, the size and cost of the downstream treatment works may be greatly reduced by the promotion of treatment within the sewer. This may prove of particular benefit in the UK, for example, where many new or upgraded plants are required in coastal areas and land use may be restricted, or especially sensitive, to the development of a treatment plant.

In situations where an existing treatment plant is overloaded, in-sewer treatment may be especially beneficial. Treatment in the sewer may not only reduce the organic loading on

the plant but also increase the treatability of the wastewater by discharging it at the plant in a truly aerobic state.

Under favourable conditions i.e. longer retention times and higher temperatures (Hemmings *et al.*, 1983), or by seeding the sewers with active biomass (Stoyer, 1970; Green *et al.*, 1985), an aerobic sewer followed by a sedimentation tank could prove a cost-effective alternative to conventional high rate carbonaceous removal facilities.

The problems of odour and corrosion associated with the formation of sulphides in the sewerage system have been widely reported (Laughlin, 1964; Boon *et al.*, 1977; Pomeroy, 1992) and much work has been undertaken solely with the intention of preventing sulphide build up. In these instances partial treatment of the wastewater has been seen merely as an additional benefit. In a situation, however, where in-sewer treatment was engineered to achieve the most practicable degree of treatment, odour and corrosion control would become significant additional benefits of the scheme .

Some work on in-sewer treatment has been undertaken in the last twenty five years which, in turn, has caused a growing interest in potential applications but there is little evidence to indicate that this work has developed into more than a handful of full-scale schemes.

This apparent indifference does not necessarily indicate that in-sewer treatment is not feasible. It should be noted that sewerage systems only grow through urbanisation and as a move away from on-site wastewater disposal methods such as septic tanks for individual houses. At the outset, the sewer is only seen as a method of transporting the

wastewater to a single point, but once a sewerage system is in place and growing, the emphasis is fixed firmly upon achieving efficient treatment at the final point of collection with the effect that the majority of collective efforts have been focused on the wastewater treatment plants themselves.

Furthermore, the improved availability of funds for more conventional treatment methods, and greater process understanding and development efforts to squeeze the existing assets, were key drivers which reduced the needs to refine in-sewer treatment (Noone, 1995).

It is also interesting to note that work in the field of in-sewer treatment has progressed in a piece-meal fashion. Even the most recent work has tended not to refer to previous studies and, as a result, there has been considerable duplication of effort.

The intention of this study is to bring together the up-to-date information in the field of in-sewer treatment, and to undertake laboratory studies to assess the general feasibility of using the sewers as a suspended growth aerobic biological reactor to achieve partial or full treatment of the wastewater during transit. Through the combination of this work, an assessment of the general feasibility of in-sewer treatment is presented and parameters influencing the in-sewer treatment have been outlined.



## CHAPTER TWO

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### LITERATURE REVIEW

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#### 2.1. INTRODUCTION

The use of sewers for the dual purpose of transportation and treatment of wastewater has been the subject of several investigations over the past 25 years. In this chapter, an attempt has been made to bring together up-to-date information in the field of in-sewer treatment. Following a summary of previous investigations, various aspects of dissolving oxygen in sewers including the oxygen requirements of the wastewater, factors affecting the dissolution of oxygen in sewers, and various methods of supplementing dissolved oxygen in pressure mains and gravity sewers are discussed.

#### 2.2. RESEARCH IN THE FIELD OF IN-SEWER TREATMENT

A brief description of various studies undertaken to investigate the use of the sewer as an aerobic treatment facility is given in the following sections.

##### 2.2.1. Gravity sewers

###### (i) Self-purification in gravity sewers

In gravity sewers, especially where the amount of oxygen dissolved by turbulence produced at junctions and other structures is sufficient to maintain aerobic conditions, a significant degree of in-sewer purification could occur (Pomeroy and Parkhurst, 1972; Pomeroy and Lofy, 1977). Pomeroy and Parkhurst (1972) observed that the BOD<sub>5</sub> in a trunk sewer, which drained a hilly area, was reduced from an average of 192 to 141 mg/l (26%) during a retention period of about 4 hours. Thomas *et al.* (1985) noted a 20% COD removal in a

sewer in which dissolved oxygen was enhanced by a high degree of ventilation and the air dissolved at falls and eddies. Raunkjaer *et al.* (1995) reported removals of 25 per cent and 14 per cent in dissolved and total COD respectively, under aerobic conditions at 15°C, in a gravity sewer having a diameter of 500 mm and a retention time of approximately 3 hours. The dissolved COD of the influent wastewater varied from 200-300 mg/l.

**(ii) Laboratory studies on in-sewer treatment in gravity sewers**

Laboratory studies, to assess the feasibility of in-sewer aerobic treatment, were originally undertaken at the University of Newcastle upon Tyne (UNUT) at the request of Anglian Water plc. These laboratory studies, which were carried out by this author during the initial stages of the work described in this dissertation, have been reported as UNUT (1994). Batch tests, undertaken to simulate the aerobic transport of domestic wastewater, showed that the addition of oxygen to raw wastewater could result in soluble COD and soluble BOD<sub>5</sub> removal efficiencies of 30 and 43% respectively, over a retention period of 3 hours. The corresponding removal efficiencies over a retention period of 6 hours averaged 46 and 78%, respectively. The influent soluble COD and soluble BOD<sub>5</sub> ranged from 152-206 and 58-74 mg/l, respectively.

Batch studies to investigate the relative contribution of suspended biomass and biofilm during the biodegradation of substrate in dual-phase systems (sewers and drains) have been presented by Cao *et al.* (1992), and Cao and Alaerts (1995). It was noted that the biofilm played a significant role in biodegradation when the true wetted area per unit volume of water was large, and the suspended biomass concentration was below a certain value (12 mg/l). The specific activity of suspended biomass with respect to the oxygen consumption rate was found to be 50 to 100 per cent of that of the biofilm.

Manandhar and Schroder (1995) used an 87.12 m long, 0.20 m deep and 0.08 m wide galvanised steel channel to investigate the possibility of treating a wastewater by circulating it to its point of origin. The slime area in the channel varied from 20.1-34.81 m<sup>2</sup>, while the inflow to the sewer was maintained at 0.36 m<sup>3</sup>/h. The average filtered COD removal in the system ranged from 49-63% at a temperature of 29-33°C, when surface aeration was the only source of dissolved oxygen. The corresponding influent COD and SS varied 46-61 mg/l, and 69-86 mg/l, respectively. The authors also developed a design equation for calculating the length of a loop sewerage system to achieve a given degree of in-sewer treatment.

### **2.2.2. In-sewer treatment in pressure mains**

The idea of using pressure pipes for in-sewer treatment appears to have emerged when the use of force main aeration for sulphide control resulted in a decrease in the BOD<sub>5</sub> of wastewater reaching a downstream treatment plant. Pomeroy (1959) noted that air injection to control sulphide in a 61 cm diameter, 8.8 km long force main, which had an average flow of 300 m<sup>3</sup>/h, reduced the average BOD<sub>5</sub> of the wastewater from 146 to 64 mg/l (i.e. 44%). Laughlin (1964), based on a study of air injection in a 1.6 km long force main which had a diameter of 76 cm and an average flow rate of 237 m<sup>3</sup>/h, reported that in addition to the control of sulphides, air injection resulted in a 20-50% reduction in BOD<sub>5</sub> of wastewater during the night when the retention period was long. No significant reduction in BOD<sub>5</sub> however, was observed during the day.

Tanaka and Takanaka (1995) studied the removal of organic matter in a force main having a retention time of 6-7 hours, with air being injected to control sulphides. Reductions of 30-55

and 30-75 per cent were noted for total and soluble BOD<sub>5</sub> respectively, at 25°C. The soluble BOD<sub>5</sub> of the wastewater at the inlet of the sewer varied from 70-240 mg/l.

With the advent of pure oxygen, the use of in-sewer oxygenation for sulphide control also became widespread. Due to higher solubility (approximately 100% oxygen as compared to 20% oxygen in air), oxygen injection enabled higher dissolved oxygen concentrations to be maintained within the sewer, which in addition to alleviating sulphide problems, resulted in a significant degree of in-sewer treatment and also improved primary sedimentation at the downstream treatment works. A typical example of such a case is an 8 km long pressure main in Bath, UK, where oxygen injection, initially used to control sulphide, led to the use of the sewer for the reduction of load on a 40% overloaded biological filtration plant. This Bath study has been described by Pullin (1977), Boon *et al.* (1977), Newcombe *et al.* (1979), Carne *et al.* (1982) and Toms and Booth (1982).

The Bath pressure main had an average retention time of about 3 hours. The oxygen requirement was estimated to be 14 mg/l.h and 700 mg/m<sup>2</sup>.h for the wastewater and slime layer respectively. Oxygen was injected directly into the sewer on the discharge side of the pump using a BOC Primox injector (Figure 2.5). The removal of BOD<sub>5</sub> settled wastewater, which included the removal of soluble BOD<sub>5</sub> and the effect of improved settleability, averaged 68 kg/h and 26 kg/h at mean wastewater temperatures of 21°C and 11°C respectively. These removal figures represented reductions of 50 per cent and 30 per cent respectively in the daily BOD<sub>5</sub> load of settled wastewater. Oxygen injection also resulted in the elimination of sulphide problems and reduced surge pressures. Although oxygen injection required an increase by approximately 50 per cent in pumping energy due to gas locking at

high points in the main, an economic appraisal of the system showed that the use of in-main oxygen injection with the dual objectives of controlling sulphides and in-sewer treatment would be economical when compared to the extension of the existing plant by conventional methods and when using chemicals for sulphide control.

In Australia, research in the field of in-sewer treatment was pioneered by Commonwealth Industrial Gases (CIG) Ltd. After laboratory- and pilot-scale studies (Greenfield *et al.*, 1980; Hutchinson, 1981), a full-scale in-sewer treatment facility aimed at achieving complete carbonaceous treatment of wastewater was commissioned at Boulder Bay (Hemmings *et al.*, 1983). It consisted of a two-step feed, 6 km long sewer having three pressure mains with step-flow increases of 35 and 40%. The total retention time in the sewer at dry weather flow was 17 hours in summer (due to holiday population increases) and 28 hours in winter, with step flows joining the main flow after retention times of 6.1 and 11.4 hours in summer and 9.1 and 17.5 hours in winter, respectively. The oxygen uptake rate varied from 14 mg/l.h at 24-28°C, and 3-5 mg/l.h at 16-20°C. A primary settling tank was installed at the end of the sewer, with the effluent being discharged to the ocean. Oxygen was dissolved at three points along the sewer using direct injection at the first point and CIG multiple U-tube dissolvers (Figure 2.6) at the others. During a 12-months trial, the Boulder Bay in-sewer treatment system was found to achieve an average BOD<sub>5</sub>/NFR (non filterable residue) effluent of 69/65 mg/l in the non-holiday winter period and 76/69 mg/l in the higher loading summer period. The influent BOD<sub>5</sub> and NFR ranged from 130-184 mg/l and 241-314 mg/l, respectively.

Hemmings *et al.* (1983) claimed that the in-sewer treatment could be linked to an ultra high-rate activated sludge process because it relies on a high growth rate of micro-organisms. In a

conventional activated sludge process, growth rate is low due to limiting substrate conditions, in spite of high MLSS levels. In an in-sewer aerobic treatment system, high substrate concentrations would be maintained along the most of the length of the reactor, resulting in a high cell growth which would compensate for any low cell population. Presently, the Bath system is not in operation while the Boulder Bay system is used as a sulphide control facility (Ogston, 1994). The main reasons that the above systems were abandoned were the improved availability of funds for more conventional treatment methods, greater process understanding, and development efforts to squeeze the existing assets (Noone, 1995).

The use of nitrates for control of sulphides has also been reported to result in a significant degree of in-sewer treatment in pressure mains. Bentzen *et al.* (1995), who injected nitrates in a rising main with an average retention time of 5.4 hours, found that controlled dosing of nitrates, in addition to effective elimination of sulphides, resulted in a soluble BOD<sub>5</sub> removal of 35 per cent at a temperature of 12-22°C. The soluble BOD<sub>5</sub> of the wastewater at the inlet of the main averaged 137 mg/l.

### **2.2.3. Enhanced treatment in sewers**

When compared with the activated sludge process, which contains a high number of active microorganisms, the active biomass is low in raw wastewater. The treatment of wastewater in sewers, under normal conditions may therefore require significantly higher retention periods (Shaw, 1981). In-sewer treatment however, could be improved by seeding the sewers with activated sludge to increase its active biomass concentration (Pomeroy and Lofy, 1977; Shaw, 1981). The above concept has been studied by a number of researchers as described below.

The earliest investigations on the use of pressure pipe for enhanced treatment of wastewater were undertaken at the Irvine Ranch Water District (IRWD), California, and have been documented by Stoyer (1970), Stoyer and Scherfig (1972) and Anon. (1973). Laboratory- and subsequently large-scale pilot studies were conducted to investigate the possibility of using a long pressure main employing activated sludge addition at the head of the main, and oxygen or air injection at several points along the main (Figure 2.1).

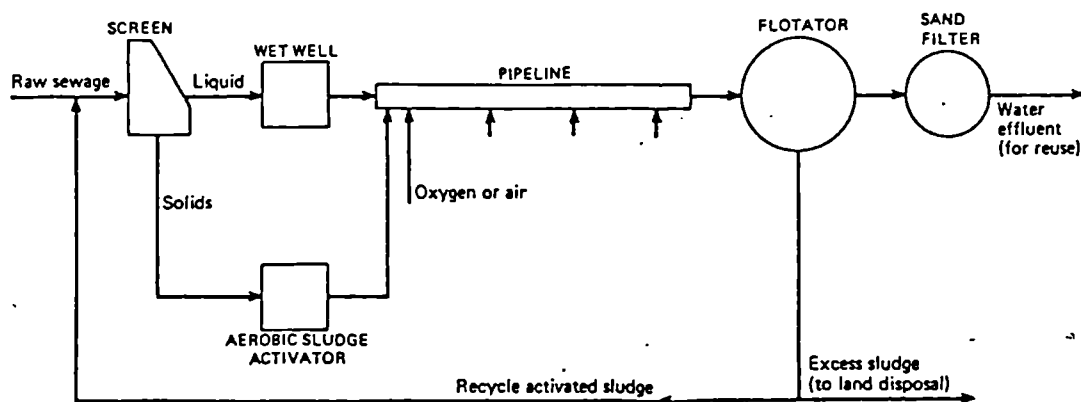


Figure 2.1- The Irvine Ranch Water District PPT system (Anon., 1973)

The main objective of the IRWD system referred to as the "pressure pipe treatment (PPT) system" was to use the pressure main for both treatment and transportation of the wastewater so that it could be reused upstream. Pilot-scale studies on two mains having lengths of 5.1 km and 4.57 km and diameters of 61 cm and 10 cm, respectively, showed that an average  $BOD_5$  removal efficiency of 90 per cent or more was possible, within a retention time of about 6 hours at 27.5°C. The influent  $BOD_5$  averaged 125 mg/l. The effluent quality was comparable to that of a conventional activated sludge plant. Based on a theoretical quantitative comparison of completely mixed and plug flow reactors, and the results of pilot-scale studies, it was argued that a residence time of about 1 hour would be

sufficient for pipeline treatment as compared with the conventional completely mixed process, due to the acceleration of BOD<sub>5</sub> removal as a result of high initial BOD<sub>5</sub> values.

The PPT system, however, did not get much beyond large-scale pilot plant testing (Hancock, 1994). According to Scherfig (1994), bench- and small-scale pilot plant testing showed encouraging results but significant problems were encountered with solids separation (using flotation) during large-scale pilot testing. The main reason that the project could not be pursued further, however, was that the demand for the reclaimed water increased dramatically in the area between the downstream treatment plant and the upstream water reservoir. "The increase in demand made the pipeline very useful as both a direct conveyance line and also as a back-bone line in the reclaimed water distribution system" Scherfig (1994).

Pilot-scale studies undertaken to investigate the use of a plug-flow pressure aeration basin followed by a conventional secondary clarifier, for the treatment of wastewater have been described by Moser *et al.* (1977). The aeration basin was constructed from a reinforced asbestos concrete tube with a diameter of 1.5 m and a length of 27 m, and was designed for a pressure of 10 bars. At a loading rate of 3 kg BOD<sub>5</sub>/m<sup>3</sup>.day and at a sludge concentration of 4-5 kg MLSS/m<sup>3</sup> in the basin, the effluent BOD<sub>5</sub> was found to be 25 mg/l or lower. The BOD<sub>5</sub> of the feed was up to 280 mg/l. The corresponding sludge volume index varied from 50 to 60 ml/g. It was concluded that the pressure plug flow aeration basin would give excellent BOD<sub>5</sub> removal and smooth operation.

Laboratory studies to investigate the possibility of using a U-shaped sewerage system in Greater Tel Aviv, Israel, for the treatment of wastewater have been presented by Green and



Shelef (1980) and Green *et al.* (1985). A batch reactor was used to simulate the 39 km long step-fed sewer with a retention time of 10 hours. The study considered the sewer as a step-fed plug flow reactor with sludge being recycled to the head of the sewer from the downstream treatment plant and air injected at several points along the sewer. The removal efficiencies were investigated at two initial VSS concentrations (after the first wastewater injection) i.e. 4760 mg/l and 1430 mg/l. The corresponding final VSS concentrations (as a result of dilution due to four feedings and biomass growth) were found to be 1000 and 500 mg/l. The corresponding influent soluble CODs of the synthetic wastewater (weighted average of all the steps) averaged 590 and 667 mg/l respectively. The dissolved COD and BOD<sub>5</sub> removals averaged 79 and 93% respectively at an initial biomass concentration of 4760 mg VSS/l and at temperatures of 20-23°C. The corresponding COD removal for an initial biomass concentration of 1430 mg VSS/l averaged 73%.

The laboratory studies undertaken during the earlier stage of the current study (UNUT, 1994) also showed that addition of activated sludge to raw wastewater at a concentration of 500 mg VSS/l could result in soluble COD and BOD<sub>5</sub> removal efficiencies of 52 and 78%, respectively over a retention period of 3 hours. The corresponding removal efficiencies over a retention period of 6 hours averaged 64 and 93%, respectively.

#### **2.2.4. Models for in-sewer treatment**

A number of models have been developed to describe various processes occurring in sewers e.g. degradation of organic matter in suspension and biofilm, hydrolysis of organic matter and biomass growth. A brief description of these models follows.

**(i) Gravity sewers**

A reactive sewer model to simulate the flow routing through the sewer, oxygen transfer through the air-water interface, and biological reactions occurring under aerobic, anoxic and anaerobic conditions, has been presented by Gall *et al.* (1995). The model considers the sewer as a series of completely mixed tanks while the aerobic biological reactions are described in a way similar to that in General Activated Sludge Model (Dold, 1990).

The MOUSE TRAP model, which describes surface run off quality, sediment transport, advection and dispersion, and water quality in sewers has been discussed by Garsdal *et al.* (1995). The main processes incorporated in the water quality part of the above model include the degradation of organic matter, bacterial fate, surface aeration, and oxygen demand from eroded sewer sediments.

Ozer and Kasigra (1995) used a heterogen system, which takes into account the substrate flux rate through the biofilm surface as well as diffusion and biochemical reactions in concomitance, to develop a model to predict the substrate removal in gravity sewers. The model did not consider the liquid phase and was based on the assumption that substrate transfer in sewer is characterised by molecular diffusion and substrate consumption which converges to the Monod Equation. Based on the results of the model, Ozer and Kasigra (1995) showed that a 23 km long sewer of 100 cm diameter would be required for a 70 per cent COD reduction of a weak wastewater (having soluble COD=170 mg/l) with an assumed velocity of 0.5 m/s.

## **(ii) Pressure mains**

Koch and Zandi (1973) presented a theoretical model to describe the reactions occurring in pressure pipelines to be used for aerobic biological treatment. Considering the pipeline reactor as a series of completely mixed reactors, equations were developed to describe cell growth, substrate utilisation, dissolved and gaseous oxygen, and pressure losses along the reactor. Using the model, Koch and Zandi (1973) found that a 43 km length (retention time 5.8 hours) of pressure pipeline with either three air injections, or a single oxygen injection, would be required to achieve a 30 per cent BOD<sub>5</sub> reduction using a seed concentration of 10 mg/l. A sensitivity analysis of the model showed that increasing the seed concentration by a multiple of 5 and 10 would result in a decrease in the length of the pipeline required to achieve a 30 per cent reduction by factors of one-third and one-fifth, respectively. The oxygen requirements, however, would remain the same but the spacing of the aerators would be decreased.

In addition to the models described above, models dealing with various aspects of in-sewer treatment i.e. oxygen consumption in dual-phase systems (Cao, 1994), and oxygen transfer in sewers (Jensen, 1995) have also been developed.

## **2.3. OXYGEN REQUIREMENTS OF DOMESTIC WASTEWATER**

The critical dissolved oxygen (DO) concentration required for an aerobic process is generally thought to be about 0.5 mg/l although a higher value (1 mg/l) is generally considered more acceptable. A DO level of greater than 1 mg/l has been recommended to be maintained in sewers to prevent the formation of sulphides (Boon and Lister, 1975; Hollerbach, 1985; USEPA, 1985). Although the oxidation process has been found to be independent of the DO

concentration above the critical value (Rickard and Gaudy, 1968; Kalinski, 1971; Pomeroy and Parkhurst, 1972), a concentration of 2 mg/l is usually used as a design value for aerobic biological reactors (Reynolds, 1982). The amount of oxygen to be added to flowing wastewater to maintain aerobic conditions is governed by the oxygen utilisation rates of suspended biomass, and the biofilm or slime layer on the wetted perimeter of the sewer.

### **2.3.1. Oxygen uptake rate of suspended biomass**

Under aerobic conditions, the oxygen utilisation rate of suspended biomass in the wastewater (SOUR) depends upon the age of wastewater, amount of biomass, concentration of soluble organic matter, temperature of wastewater, concentrations of substances inhibitory to biochemical oxidation and pH (Boon and Lister, 1975; Boon *et al.*, 1979; Newcombe *et al.* 1979; Simpson, 1979; Vivona, 1979; Ozer and Kasigra, 1995).

Pomeroy and Parkhurst (1972) undertook extensive field and laboratory studies to investigate the oxygen uptake rate of wastewater in the Los Angeles County Sanitation District sewerage systems. It was found that the SOUR varied widely and did not bear any correlation with the BOD<sub>5</sub> of the wastewater. Under aerobic conditions, domestic wastewater was found to have a low SOUR (2 to 3 mg/l.h) near its point of origin which increased for several hours reaching as high as 20 mg/l.h, after which it declined. The duration at which peak SOUR occurred varied greatly, being higher (6 to 10 hours) for a fresh domestic wastewater and much less (2 hours) for samples collected after being transported for about 10 hours under low DO conditions. Several wastewaters also exhibited a pronounced secondary SOUR peak following the main peak. The reasons for this secondary peak could not be specified. The SOUR tended to be lower for small sewers than in large ones. Tributary sewers were found

to bring in wastewater that had a lower SOUR than the main line flow, producing a saw-tooth profile of SOUR.

Boon *et al.* (1977) also reported SOUR profiles of fresh domestic wastewater, similar to that observed by Pomeroy and Parkhurst (1972). Marwood (1984) as quoted by Dack and Nadebaum (1989) described the SOUR of an oxygenated, previously anaerobic wastewater as a three phase process i.e. the first phase in which rapid oxidation of sulphide, and other minor chemical oxidation and biological uptake occurs, the second phase characterised by a low oxygen uptake which is then followed by the third phase in which SOUR increases rapidly due to acclimatisation of facultative aerobic bacteria. Matos and de Sousa (1991) suggest that an increase in SOUR of wastewater with age may be due to sloughing of biologically active slimes, and gradual transformation of complex organics into easily biodegradable compounds. A decrease in SOUR after the peak value has been attributed to the depletion of readily biodegradable organic matter (Lokkegaard *et al.*, 1995).

Ozer and Kasigra (1995), on the basis of their laboratory studies, reported that the SOUR is strongly affected by substrate concentration and suspended biomass. They developed the following empirical equation showing correlation between SOUR and soluble organic matter.

$$RES = 0.077 + 0.00064 S \quad (2.1)$$

Where; RES (mg/l.min) and S (mg/l) represent the respiration rate of the wastewater and the biodegradable part of the filtered COD (mg/l) respectively.

### 2.3.2. Biofilm (or slime layer) oxygen uptake rate

The oxygen uptake rate of the biofilm on the wetted perimeter of the sewer (BOUR) depends upon the amount of readily biodegradable organic matter in the wastewater, dissolved oxygen (DO) concentration of the wastewater, temperature, velocity, and specific surface area of the sewer. Each of these parameters is discussed below.

Thistlethwayte (1972) states that the slime activity would be higher for a strong wastewater due to greater availability of nutrients. In contrast to this, Boon (1995) reports BOUR to be independent of  $BOD_5$  of the wastewater (within a range of 90-780 mg/l). BOUR increases with temperature due to increased slime activity, and reduced kinematic viscosity of the wastewater which in turn decreases the thickness of the boundary layer thus increasing the diffusion of nutrients to the biofilm (Thistlethwayte, 1972).

Pomeroy and Purkhurst (1973), on the basis of their field studies, report that in sewers in which slime is an effective oxygen sink, the BOUR varies directly in proportion to the oxygen concentration (Equation 2.2). It is however, indicated that the BOUR would not be proportional to DO in well aerated streams of low oxygen demand.

$$BOUR = 5.3O_2 \frac{\sqrt{SU}}{r} \quad (2.2)$$

Where: BOUR = biofilm oxygen uptake rate (mg/l.h),

$O_2$  = dissolved oxygen concentration (mg/l),

S = slope of energy line (m/m),

$U$  = velocity (m/sec), and

$r$  = is the hydraulic radius of the stream (m).

On the other hand however, Boon *et al.*, (1977) argue that under aerobic conditions, the BOUR remains almost constant in sewers, probably due to more readily available DO and substrate level. Matos and de Sousa (1990) also concluded that BOUR did not show any evident correlation with the DO concentration in small sewers with relatively low flows and relatively high DO concentrations (above 3 mg/l). Nielsen *et al.* (1992) reported that the BOUR of the laboratory biofilms grown in biofilm reactors designed to simulate the sewer slimes, was almost independent of DO concentration above 0.5 mg O<sub>2</sub>/l.

The BOUR increases with an increase in the velocity of wastewater (Pomeroy and Parkhurst, 1972; Matos and de Sousa, 1991; Nielsen *et al.*, 1992), probably due to increased transfer of organic matter by molecular diffusion into the slime layer, caused by increased turbulence at higher velocities. Pomeroy and Parkhurst (1972) conclude that slimes are not likely to be absent in sewers unless the velocity exceeds 2.2 m/sec or abrasive material is present.

Generally for pressure mains, slime activity is assumed uniform over the whole surface area of the pipe line. In gravity sewers however, the active slime area varies with diurnal variations in wastewater level due to changes in flow. In this case, an estimate of active slime area could be made based on the minimum, average and maximum value of the flow rate (Newcombe *et al.*, 1979). Considering the variations in wall shear, diameter and alignments, pipe jointing details, and wastewater level, Thistlethwayte (1972) adopted an active slime growth of 15% of the wetted surface area as a design criterion for sulphide control. He also

presented design charts for assessing approximate values of slime area under various influencing factors.

### 2.3.3. Total oxygen requirements

The total oxygen uptake rate of wastewater in a particular sewer would be the sum of the SOUR and the BOUR. The daily total oxygen requirement would be determined by the wastewater flow, while the maximum DO concentration to be achieved at a given point along the sewer would depend upon the retention time in the sewer at the minimum flow.

To obtain a reliable estimate of oxygen input for design purposes, it has been recommended that wastewater should be tested several times for diurnal as well as seasonal sulphide concentrations, oxygen uptake rate, and rate of sulphide oxidation (US EPA, 1974; Newcombe *et al.*, 1979; Wernicke, 1987). Samples must be tested immediately after collection to have a better simulation of real conditions in the sewer. A detailed procedure for estimating the total oxygen requirements for sulphide control has been presented by USEPA (1985).

Boon and Lister (1975) have proposed the following equation for calculating the DO concentration to be introduced at the bottom of a rising main, to maintain aerobic conditions.

$$C_o = \left( SOUR + \frac{2.8}{d} \right) \frac{\pi d^2 l}{4 Q_h} 1.07^{T-15} \quad (2.3)$$



Where:  $C_o$  = dissolved oxygen concentration to be introduced at a point (mg/l)  
 $SOUR$  = oxygen uptake rate of suspended biomass (mg/l.h)  
 $d$  = diameter of the rising main (m)  
 $l$  = length of the rising main  
 $Q_h$  = average flow rate of the wastewater ( $m^3/h$ )  
 $T$  = temperature of the wastewater ( $^{\circ}C$ ) within the range of 10-20 $^{\circ}C$ .

Equation 2.3, which has been developed from empirical data (Kite and Garrett, 1983), assumes a BOUR of 700  $mg/m^2$  of the wetted area of the sewer per hour, at 15 $^{\circ}C$ .

According to Boon (1995), the average SOUR of a typical domestic wastewater ( $BOD_5$  about 400 mg/l) would be about 14 mg/l.h at 15 $^{\circ}C$ . The corresponding value for a wastewater containing a significant portion of industrial discharges or having a significantly lower  $BOD_5$  than domestic wastewater, would be 6 mg/l.h. In contrast, Speece *et al.* (1990), have reported SOUR and BOUR values of 10 and 4 mg/l.h, respectively at 20 $^{\circ}C$ .

From the oxygen uptake rates given above, it appears that the total oxygen requirement for sewers with a long retention time, especially at higher temperature, may be very high. In this case, it may not be economically feasible to dissolve such a high DO concentration at a single point and hence injection facilities would have to be located at many points along the sewer.

## 2.4. SOLUBILITY OF OXYGEN IN SEWERS

The maximum dissolved oxygen concentration that can be achieved in the flowing wastewater downstream from an injection point depends upon the pressure and temperature of wastewater, concentration of the dissolved impurities in the wastewater and the total uptake rate of oxygen along the sewer (Boon *et al.*, 1977).

**Table 2.1- Solubility (mg/l) of atmospheric and pure oxygen in pure water**

Temperature	1 atm		2 atm		5 atm		10 atm	
°C	AO <sup>a</sup>	PO <sup>b</sup>	AO	PO	AO	PO	AO	PO
10	11.3	54	22.4	107	56	268	112.4	537
20	9.2	43	18.6	87	46.6	217	92.9	434
30	7.6	36	15.2	72	37.8	179	75.8	359

Notes: <sup>a</sup> AO = atmospheric oxygen (Sawyer *et al.*, 1994)

<sup>b</sup> PO = pure oxygen (Pomeroy and Lofy, 1977)

At any given temperature, the solubility of oxygen in wastewater increases with an increase in atmospheric pressure. Therefore, more efficient aeration or oxygenation can be achieved in pressure mains than in gravity sewers where the flow is under atmospheric pressure. The solubility of oxygen decreases with an increase in temperature. Table 2.1 shows the solubility of pure and atmospheric oxygen in pure water at different temperatures and pressures.

The solubility of oxygen is not significantly affected by the dissolved impurities in wastewater except nitrogen (Pomeroy and Lofy, 1977). Raising the dissolved oxygen content of wastewater by air injection increases the dissolved nitrogen in wastewater which interferes with the dissolution of further oxygen thus reducing the efficiency of dissolution. Although the use of pure oxygen helps alleviate this problem, nitrogen interference will reduce the transfer efficiency at higher DO concentrations, especially under ambient pressures.

Wastewater contains dissolved nitrogen in an amount equal to its air saturation concentration (approximately 15 mg/l at 1 atm and 20°C). At normal wastewater temperatures, the amount of pure oxygen that would completely dissolve at equilibrium with atmospheric pressure is the same as that for water in equilibrium with air (9.2 mg/l), because the gas phase must consist of 79% nitrogen. If attempts are made to achieve higher DO concentrations by applying pure oxygen, some oxygen will remain undissolved, and some nitrogen will strip out of solution.

The stripping of nitrogen causes two problems (Speece *et al.*, 1990); (i) it decreases the oxygen transfer rate because of the dilution of oxygen concentration in the gas phase by the stripped nitrogen, and (ii) it reduces the oxygen-absorption efficiency potential because the nitrogen diluted oxygen must be wasted if higher DO concentrations are to be achieved. Raising the DO to 30 mg/l at 25 °C and 1 atmospheric pressure by single injection, for instance, will result in 29% wastage of oxygen, although the solubility of oxygen is about 40 mg/l (Pomeroy and Lofy, 1977). This wastage could be minimised if oxygen is supplied in more than one step or by removing the dissolved nitrogen from wastewater by nitrogen stripping (Charnock, 1979; Shaw, 1980).

The nitrogen stripping potential of water decreases with an increase in pressure (Speece *et al.*, 1990). At a pressure of 4 atmospheres, for example, the gas phase must consist of 20% nitrogen, as compared to 79% at ambient pressure. The saturation oxygen concentration at 4 atmospheres will be 141 mg/l DO (80% oxygen composition), while the corresponding value for ambient pressure is 9.2 mg/l (20% oxygen composition).

The oxygen transfer efficiency in pressure mains, in addition to the factors discussed above, also depends upon the bubble diameter, length and diameter of the pipe and velocity in the main (Cadee, 1984; Sowards and Carpenter, 1989).

## **2.5. IN-SEWER AERATION**

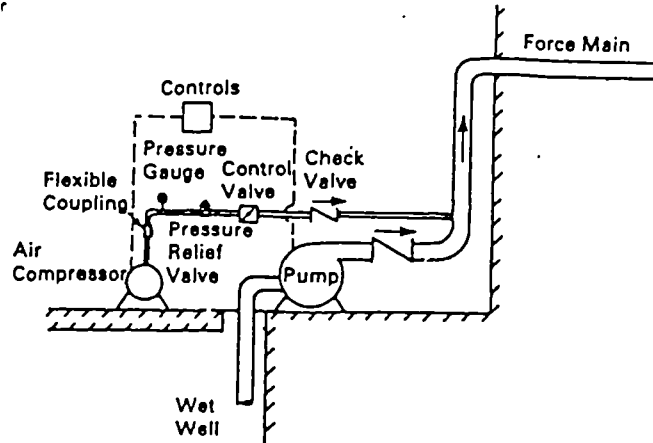
### **2.5.1. Pressure mains**

Air has been used as an effective method for controlling sulphides in pressure mains since 1942 (Pomeroy, 1959). Noticeable BOD<sub>5</sub> reductions have been reported when air was injected for sulphide control (Section 2.2.2). A brief description of various in-sewer aeration methods used for pressure mains is given in the following sections.

#### **(i) Direct injection of compressed air**

The direct injection of compressed air is the most widely used in-sewer aeration method for controlling sulphides in pressure mains (Laughlin, 1964; Vivona, 1979, 1980; Rudolph, 1981; Vivona and Whalen, 1982; Kameda and Odauchi, 1984; Bacquet *et al.*, 1992; Tanaka and Takenake, 1995). This method (Figure 2.2) involves injecting compressed air directly in the main at a low point. In a continuously upward sloping pipe,

turbulence produced by the bubbles passing up the pipe results in efficient dissolution of oxygen.



**Figure 2.2-Direct injection of compressed air (USEPA, 1985).**

Air injection is not always feasible especially for a nearly level main or where the sewer has an irregular profile (US EPA, 1974; Pomeroy, 1992). In a flat main the air moves easily and turbulence produced is not enough to achieve sufficient oxygen dissolution. In a pressure main with irregular profile, the injection of air results in excessive gas locking problems (Section 2.7.1).

## **(ii) U-tube aeration**

U-tube aeration (Figure 2.3) has been used as an efficient method for controlling sulphides in pressure mains (Mitchell, 1973; USEPA, 1974). As shown in Figure 2.3, air is introduced in fluid in the descending leg of the U-tube either through a venturi aspirator placed in a pipeline leading to the U-tube, or by injecting compressed air through a diffuser collar. Injection of air under pressure is necessary for situations where the force main is nearly level and the discharge pressure is low (USEPA, 1974). The U-tube produces efficient dissolution of oxygen due to the relatively long contact time of the gas with water

and increased hydrostatic pressure caused at the bottom of the U-tube due to the water column.

Various operating parameters of the U-tube include the air/water ratio, DO concentration in the inlet, depth of U-tube, water velocity through U-tube, and disperser depth, and have been discussed at length by Speece *et al.* (1969), Speece and Orosco (1970), Mitchell, (1973), Speece *et al.* (1981) and USEPA (1985). Although the use of U-tube aeration has been successful for controlling sulphides in pressure mains, it is thought unlikely that U-tubes could be used for the incremental increase in DO that would be required by biological treatment (Pomeroy and Lofy, 1977).

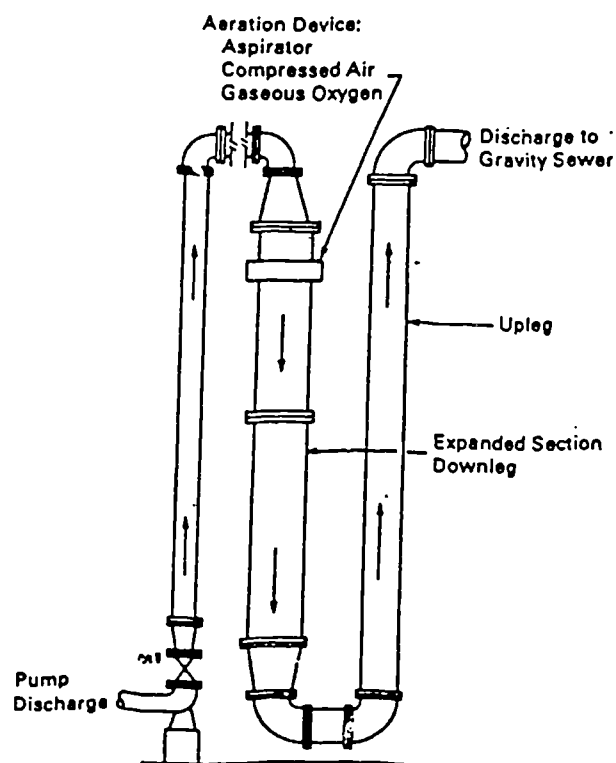


Figure 2.3-U-tube aeration (USEPA, 1985).

### **(iii) Air lifts**

An air lift is similar to a U-tube except the air is injected into the rising leg. It has been indicated that air lifts, if used for pumping of wastewater where there is relatively low lift, would add to the dissolved oxygen supply due to the energy dissipated by the rising air through the wastewater (Pomeroy and Lofy, 1977). Although, the pumping efficiency of air lift is low, it would appear to be improved if air lift is looked upon as an air pumping and oxygen dissolving device. In relatively flat sewers, a series of low-lift stations using air lifts would achieve DO concentrations sufficient to control sulphides and accomplish significant in-sewer purification. Also such a system may be economical compared with conventional pumping stations (Pomeroy and Lofy, 1977).

### **(iv) Venturi aspirators**

A venturi aspirator (Figure 2.4) consists of an inlet convergent nozzle, a throat, and a divergent barrel. Liquid flowing through a high velocity nozzle creates suction at the discharge side, thus allowing air to be drawn in from the air entraining pipe. In the throat, where velocity is maximum and pressure is minimum, the entrained air is combined with the liquid and is compressed to the discharge pressure by the momentum derived from the pumped jet. The intimate mixture of air and liquid is ejected via the divergent barrel into the bulk of the liquid where high turbulence causes intimate mixing of air and water.

The venturi aspirator has been used for sulphide control in force mains and lift stations in the USA (USEPA, 1985). In the UK, there appears to be no documentary evidence regarding the use of venturi aspirators for sulphide control in force mains. However a venturi system marketed by Tom Maguire and Company Ltd. under the name VO<sub>2</sub>, could possibly be used for this purpose. VO<sub>2</sub> has been used for the treatment of sludges, and

domestic and industrial wastes. An oxygen transfer efficiency of up to 30% has been claimed (ERC, 1985; Tom Maguire, 1986).

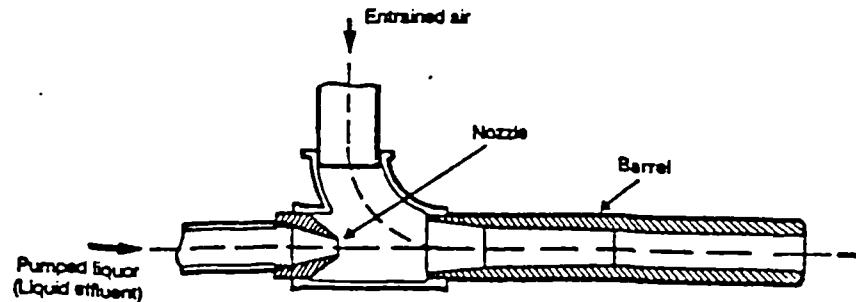


Figure 2.4-Tom Maguire VO<sub>2</sub> venturi aspirator (ERC, 1986).

### 2.5.2. Gravity sewers

A comprehensive review of various sources of dissolved oxygen and tentative in-sewer aeration devices for gravity sewers, was presented by Pomeroy and Lofy (1977), and is summarised as follows:

#### (i) Surface aeration and ventilation

The main sources of oxygen supply in gravity sewers include normal surface aeration, and increased surface aeration at points of high turbulence. The amount of oxygen dissolved in the water stream through surface aeration is proportional to  $(su)^{3/8}b$ , where  $s$  is the slope of the energy line of the stream (m/m),  $u$  is the velocity (m/sec) and,  $b$  is the surface width of the stream (m). In very small sewers with swift flow, the amount of oxygen dissolved may be sufficient to maintain aerobic conditions. However, for a sewer more than a few centimeters deep in warm climates or more than 1 m deep where wastewater temperatures are low, oxygen supply from surface aeration will be insignificant. Considering the fact that, in large systems the wastewater is collected into larger trunk sewers with flatter slopes and deeper flows, the authors concluded that normal surface aeration will not



provide sufficient oxygen for in-sewer treatment. It is also concluded that ventilation is not a realistic method of supplementing DO for biological treatment.

#### **(ii) Water falls**

Laboratory experiments designed to study aeration in gravity sewers from a waterfall, revealed that greater oxygen absorption could be achieved by dissipating a part of the available difference between two points along a proposed sewer in a fall or a series of falls, as compared with providing steep slopes and higher velocities. It is also illustrated that a series of low falls rather than one high fall would be more effective provided there is sufficient time for oxygen depletion between the falls.

#### **(iii) Aeration at the points of high turbulence**

Pomeroy and Lofy (1977) conclude that junctions, drops, hydraulic jumps or other such points of turbulence will not provide sufficient oxygen to accomplish a significant degree of biological treatment in large trunk sewers. In smaller sewers however, the oxygen supply from these sources may be quite significant, and hence efforts should be made to maximise this effect during the design of a system.

#### **(iv) In-sewer aeration devices**

A number of tentative in-sewer aeration devices, including surface aerators, jet aeration, venturi aspirators and diffused aeration has been considered by Pomeroy and Lofy (1977). The use of mechanical aerators is discounted as their installation within a sewer will not only be difficult, but costly. In addition, as each unit only provides a small increase in dissolved oxygen, many aerators would be required. The use of jet nozzles for in-sewer aeration would

be more applicable and hence deserves further investigation. Venturi aspirators would not be favoured unless specific advantages are demonstrated in the future.

Pomeroy and Lofy (1977) concluded that the diffused aeration seems to have the highest prospects of being used for in-sewer aeration. Small or large bubble diffusers may be appropriate for installation in gravity sewers provided there is sufficient water depth for reasonable submergence of the diffuser. Small bubble diffusers may transfer up to 5 to 10% of oxygen available in the air supplied while large bubble diffusers would transfer less. Large bubble systems, however, are thought to be most appropriate because of their lower operating and maintenance requirements. The configuration for diffused aeration favoured by Pomeroy and Lofy (1977) is the use of a perforated pipe as a source of diffused air, placed in each manhole, with a compressed air supply line running parallel to the sewer. These diffusers could lie longitudinally in the sewer although a preferred arrangement would be to place the diffuser immediately downstream a weir built for this purpose, where high velocity and turbulence will help enhance the oxygen dissolution.

The use of U-tubes, air lift and pressure tank dissolvers for sulphide control in gravity sewers has also been evaluated in the USA (USEPA, 1985). These systems were installed at the end of the force mains or gravity sewers which then discharged into other gravity sewers. Data reported were not sufficient to draw any general conclusions regarding the effectiveness of these methods for supplementing DO in gravity sewers. No such installation appears to be presently in operation.

## 2.6. IN-SEWER OXYGENATION

Commercial oxygen for controlling sulphides in both the pressure mains and gravity sewers, is used world-wide. There are more than 80 full-scale systems using oxygen injection to prevent septicity in the UK (Boon, 1995). The corresponding number of installations in Australia is 157 (Ogston, 1994). The principal advantages of in-sewer oxygenation, as compared with in-sewer aeration, are outlined below:

- (i) the solubility of pure oxygen is approximately five times greater than oxygen, which enables higher concentrations of dissolved oxygen to be achieved rapidly and efficiently. This not only eliminates the sulphide problems, but also results in a significant degree of in-sewer treatment and improved primary settling characteristics (Newcombe *et al.*, 1979; Hemmings *et al.*, 1983; Wernicke, 1987). In contrast with this, air systems are characterised by low transfer efficiencies, and are unable to economically achieve high DO concentrations (Dack and Nadebaum, 1989; Sowards and Carpenter, 1989; Speece *et al.*, 1990).
- (ii) pure oxygen produces less inert gases which results in fewer gas locking problems. Air systems, on the other hand, increase the dissolved nitrogen in wastewater which not only interferes with further dissolution of oxygen, but also results in increased gas locking (Boon *et al.*, 1977),
- (iii) the use of pure oxygen for both pressure mains and gravity sewers is well established. On the other hand, in-sewer aeration has been found to be economically feasible only for continuously upward sloping pressure mains (Section 2.5.1), and
- (iv) oxygen injection systems, especially liquid oxygen, are simple and easy to install and have low maintenance requirements (Mozell, 1985).

The main advantages claimed for pure oxygen over air, from the biological treatment point of view, include the capability of meeting higher oxygen demands thus enabling high MLSS concentrations to be maintained, improved settleability (Chapman *et al.* 1976; Miller, 1978; Shelef and Green, 1980; Andreakis, 1987; Gould, 1990), less sludge production (Williamson and Nielsen, 1981; Jeffries, 1983; Gould, 1990), and greater BOD<sub>5</sub> removal rates (Crossley and Macdonald, 1972; Sidwick and Lewandowski, 1975). The last two advantages however, are not supported by many comparative studies and it has been recommended that pilot-plant studies be undertaken and an economic evaluation made before adopting a pure oxygen system for biological treatment (White, 1987; Gray, 1989).

The various disadvantages of oxygen compared with air systems include the need for sophisticated equipment for oxygen generation and the delivery of liquid oxygen, the additional cost of oxygen production, corrosion, skilled supervision (especially where the oxygen is to be produced at the site) and, an increased fire hazard. A major disadvantage of a covered, pure oxygen activated sludge process is the decrease in pH of the mixed liquor due to an accumulation of carbon dioxide in the wastewater, which in turn may cause inhibition of the nitrification process (Kalinske, 1971; Benefield and Randall, 1980). This effect however, is less serious for wastes having a high alkalinity (White, 1987).

A brief description of various in-sewer oxygenation devices is given in the following sections. Most of these techniques are developed and marketed by the International BOC group, represented by Commonwealth Industrial Gases (CIG), Afrox, Airco Industrial Gases, and BOC in Australia, South Africa, United States, and the rest of the world respectively,

while use of in-sewer oxygenation techniques developed and marketed by L' Air Liquide (France), and Linde have also been reported.

### **2.6.1. Injection of oxygen in pressure mains**

The various methods used for pure oxygen injection to pressure main are described in the following sections:

#### **(i) Direct injection**

Direct injection is the most commonly used oxygen injection method for pressure mains (Boon and Lister, 1975; Boon *et al.*, 1977; Newcombe *et al.*, 1979; Hollerbach, 1985; Mozell, 1985). This method involves direct dosing of a metered quantity of oxygen to a pressure main at the discharge side of the pump where turbulence, pressure and velocity are at its maximum thus promoting efficient transfer of oxygen. Generally the oxygen supply is interlocked with pumps so that oxygen is supplied only when pumps are in operation. The correct design of the injector and the bubble size are essential for achieving efficient dissolution of oxygen. A nozzle injector is used at locations where oxygen solution rates are critical (i.e. pumping velocities greater than 0.6 m/s and pumping pressures greater than 15 m head). Under more critical conditions (velocities less than 0.6 m/s), it has been recommended to use diffusers made of a porous material in order to promote the formation of fine bubbles (Hollerbach, 1985).

A direct injection system developed and marketed by the BOC group under the name Primox (Figure 2.5) has also been used for in-sewer treatment (Newcombe *et al.*, 1979). The principal design parameters of a CIG “Primox” in-line nozzle dissolver are given in Table 2.2.

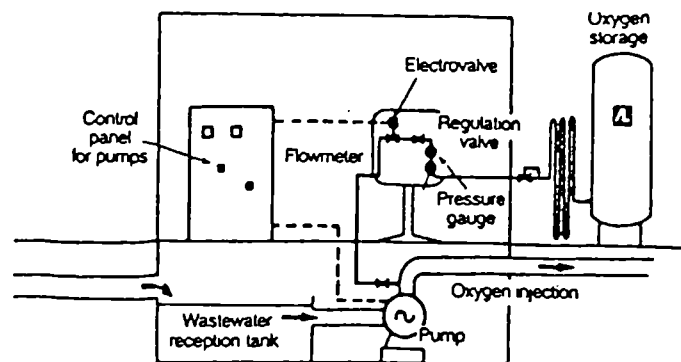


Figure 2.5. The BOC Primox oxygen injection system (Boon *et al.*, 1977).

Table 2.2-CIG "Primox" in-line dissolver design parameters (Ogston, 1994)

Nozzle velocity range (m/sec)	7-9
Minimum nozzle diameter (mm)	100
Maximum head loss (mwwg)	4.0
Oxygen transfer efficiency (% Cs)	< 60

Notes: (i) mwwg = meters of water gauge

(ii) Cs = theoretical saturation oxygen concentration at a given temperature and pressure

The main advantages of Primox oxygen injection system include its simplicity to install and operate, and less supervision and maintenance requirements (Garrett, 1984).

## (ii) U-Tube oxygenation

### (a) Single U-tube injection

Single U-tube (Section 2.5.1) oxygenation for sulphide control in sewers has been evaluated in the USA (USEPA, 1985). The U-tube provides an efficient dissolution of oxygen with low unit energy consumption. A 30 m deep U-tube employing off-gas recycle,

could result in more than 80% transfer efficiency at a through put velocity of 2.5 m/s (Speece *et al.*, 1990).

#### **(b) Vitox II Multiple U-tube dissolver**

The multiple U-tube dissolver (Figure 2.6) has been developed and marketed by the BOC group and used successfully for odour control and in-sewer treatment (Hemmings *et al.*, 1983; Cadee *et al.*, 1984; Sowards and Barkley, 1992). Its main advantages include the ability to achieve up to 75% of the saturation oxygen concentration at an oxygen utilisation efficiency of more than 90%, freedom from gas locking problems, and minimum attention and maintenance requirements because of the absence of moving parts (Cadee *et al.*, 1984; Hollerbach, 1985). Table 2.3 lists the main design criteria of the CIG U-tube dissolver.

**Table 2.3-CIG U-tube dissolver design parameters (Ogston, 1994)**

Minimum velocity: single and three stage (m/sec)	0.45
Maximum velocity: single and three stage (m/sec)	1.2
Oxygen transfer efficiency: (% Cs)	
- single stage	75
-three stage	60

The world's largest U-tube dissolver is located at Munster Pumping station, upstream of Woodman WWTP, Perth, Australia, and is quoted as being capable of dissolving more than 53 mg/l of oxygen at 30° C (Cadee *et al.*, 1984). The dimensions of this dissolver are as follows:

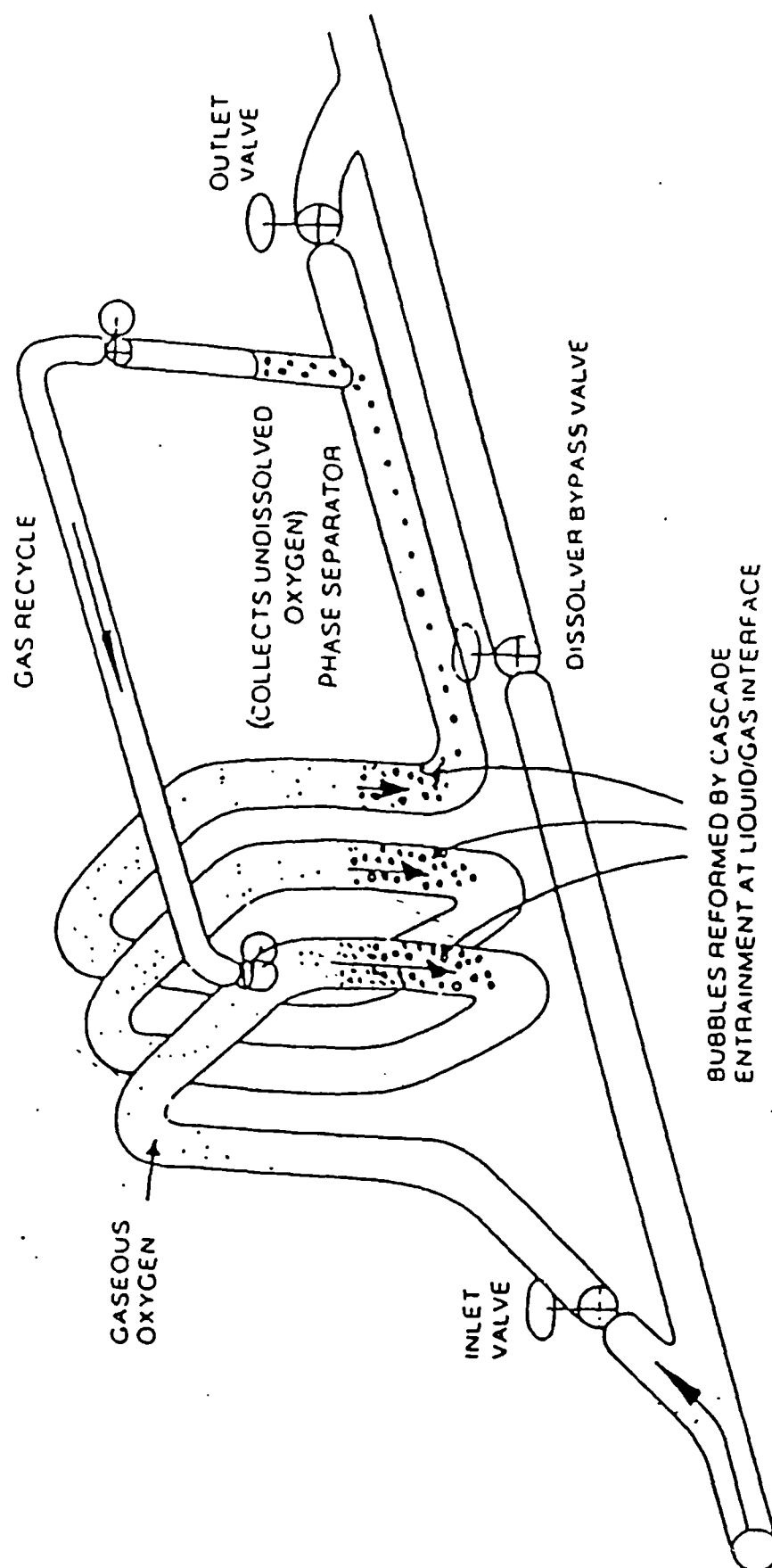


Figure 2.6. The CIG multiple U-tube dissolver (Ogston, 1994).



(i) pipe diameter = 1100 mm (ii) height = 7500 mm (iii) width = 6970 mm (iv) phase separator diameter = 2150 mm (v) phase separator length = 6270 mm (vi) oxygen return line diameter = 200 mm.

### **(iii) Side stream dissolvers**

Direct injection of pure oxygen results in efficient dissolution of oxygen if the velocity in the main is sufficient to maintain two-phase flow. For force mains, where the wastewater velocity is insufficient to maintain bubbles in suspension, use of sidestream dissolvers (Section 2.6.2) has been recommended (Speece *et al.*, 1990). Sidestream DO concentrations of up to 30-60 mg/l can be achieved depending upon the static head. Alternately, where sufficient hydrostatic head is not available, the sidestream could be pressurised up to 35 mwg using a throttling valve on the effluent of the oxygenation chamber, to achieve a sidestream DO level of 130 mg/l.

## **2.6.2. Injection of oxygen in gravity sewers**

The use of pure oxygen for increasing the DO level in gravity sewers has been found to be very successful and is now well developed. The commonly used oxygen injection techniques are described in the following sections.

### **(i) Sidestream venturi dissolver**

The most commonly used sidestream system for both pressure mains and gravity sewers is the one developed and marketed by the BOC group under the name Vitox (Forrest, 1985; Garrett and Jeffries, 1984; Dack and Nadebaum, 1989). The Vitox system (Figure 2.7) draws a portion of main flow which is then pressurised and delivered to a venturi where oxygen is

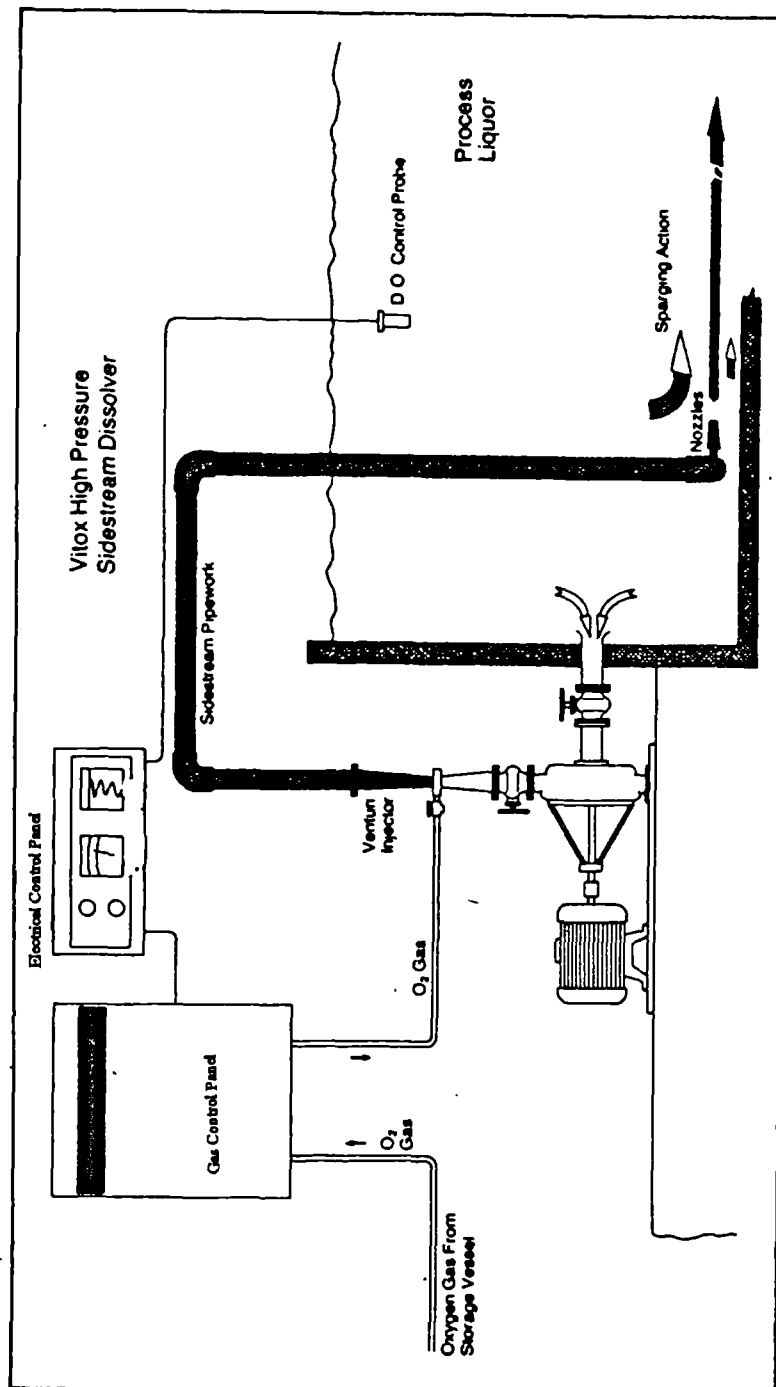


Figure 2.7. The Vitox sidestream venturi dissolver (BOC, 1994).

injected. The sidestream injected with oxygen is then returned to main flow through a high velocity nozzle. A substantial portion of oxygen is dissolved under pressure. The discharge of sidestream through the high velocity nozzle forms micro bubbles and causes rapid mixing and further dissolution. The Vitox system has been reported to achieve an oxygen transfer efficiency of 40-60% in gravity sewers (Hollerbach, 1985).

The design and operating parameters of a sidestream venturi system include the nozzle diameter, sidestream flow, pressure, and, the target DO concentration (Dack and Nadebaum, 1989; Holder and Leow, 1994). Optimum values of these parameters for a particular site are usually based on field tests. It has been shown that higher transfer efficiencies could be achieved for smaller sidestream flows and small nozzle diameters (Dack and Nadebaum, 1989).

#### **(ii) Counter-current technique**

As already discussed in Section 2.4, nitrogen stripping inhibits the efficient transfer of oxygen, especially at ambient pressures. One way to alleviate this problem is to use counter-current technique (Figure 2.8). A DO concentration of up to 30 mg/l can be achieved at ambient pressures (Shaw, 1981). A transfer efficiency of up to 70% has been reported (Hollerbach, 1985).

#### **(iii) U-tubes and multiple U-tube dissolver**

Single U-tubes (Section 2.5.1) have been successfully used for sulphide control in gravity sewers (USEPA, 1985). The Multiple U-tube dissolver described in Section 2.6.1 can also

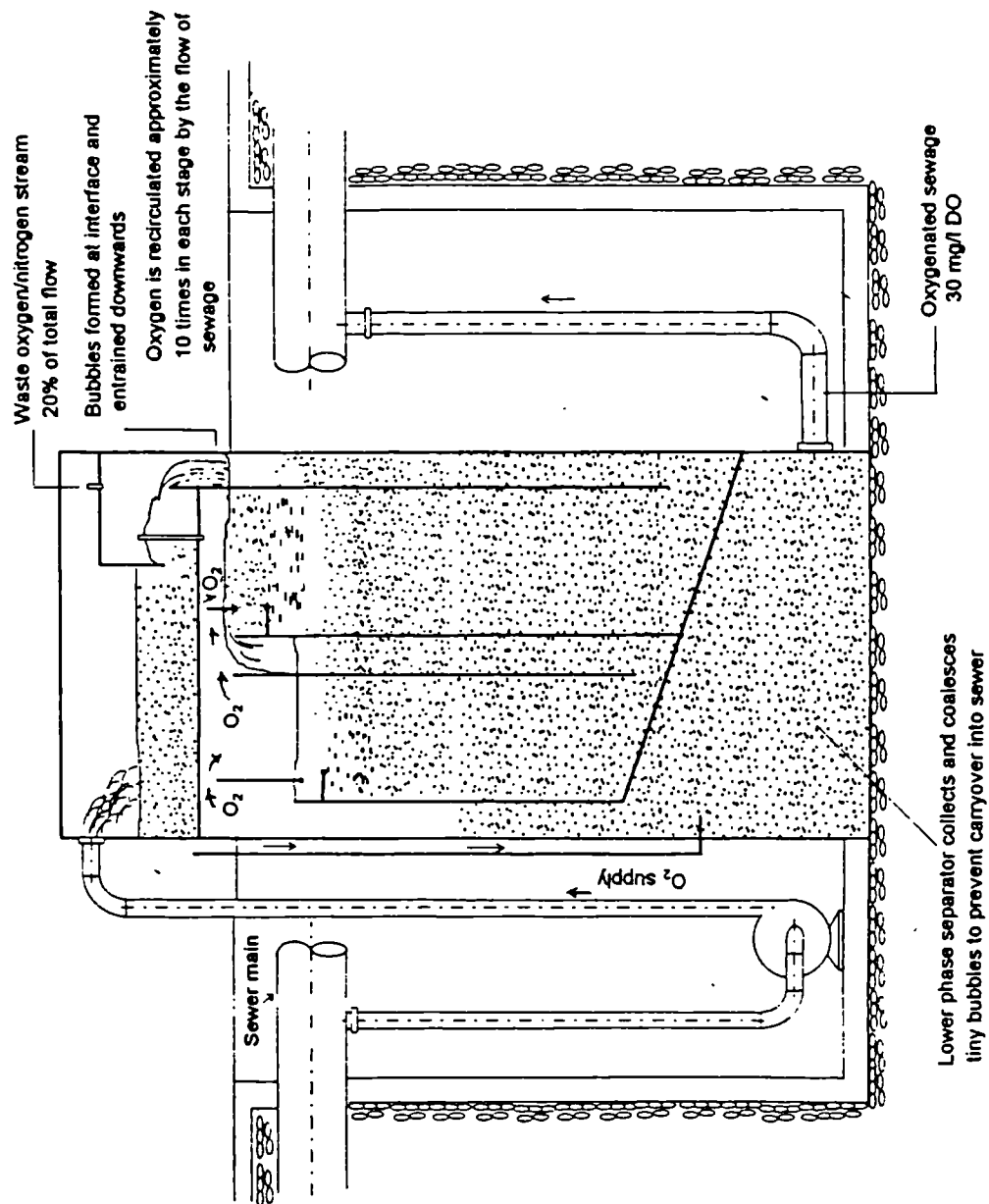


Figure 2.8-The counter current technique (Shaw, 1981).

be used for gravity sewers by employing an in-line booster pump (Dack and Nadebaum, 1989).

#### (iv) Other techniques

In addition to the various oxygen injection methods described above, the use of the Biocone system (Wernicke, 1987; Weinsaepfel and Rabergeau, 1991), and non-porous membranes (Sekoulov and Muller, 1986) for dissolving pure oxygen in both pressure mains and gravity sewers is also documented. The Biocone system is developed and marketed by L' Air Liquide and is shown in Figure 2.9.

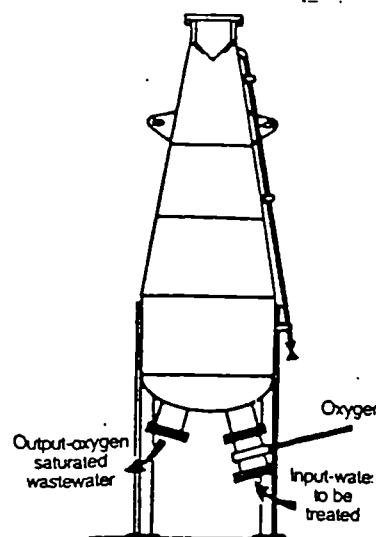


Figure 2.9-Biocone system (Mozzel, 1985).

The non porous membrane system (Figure 2.10) employs flexible thin-walled tubes of either a simple or an annular pipe arrangement. Oxygen is allowed to diffuse into wastewater under a pressure of 4 bars. A mass transfer rate of  $10 \text{ mg/m}^2\cdot\text{h}$  has been claimed. The main advantages of this system, as compared with conventional systems include no loss of oxygen or gas stripping, and freedom from clogging. Oxygen injection

into a hydraulic water fall in gravity sewers has been reported by USEPA (1985) and Speece *et al.* (1990).

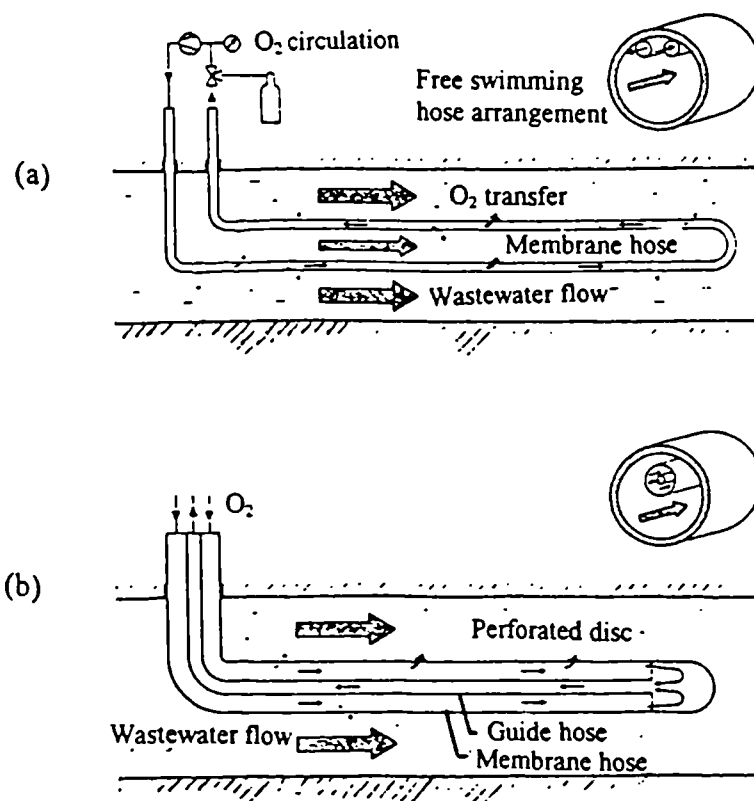


Figure 2.10-The non-porous membrane system (Sekoulov and Muller, 1986).

(a) Enclosed gas circulation; (b) Single hose system

## 2.7. ADVERSE EFFECTS OF IN-SEWER AERATION OR OXYGENATION

### 2.7.1. Gas locking

As discussed in Section 2.4, attempts to achieve higher concentrations of dissolved oxygen in sewers by injecting air or oxygen result in the production of residual gases. In a pressure main of even gradient, these undissolved gases are carried along the pipe soffit at approximately 80% of flow velocity (Carne *et al.*, 1982). However, in a pressure main of irregular profile, the bouancy of the residual gases tends to resist flow in the downward gradient, thus causing accumulation of bubbles at high points i.e. "gas-locking" (USEPA,

1974; Dawson, 1980; Wernicke, 1987). The degree of gas locking is mainly affected by the rate of gas injection, degree of oxygen dispersion at the injection points, wastewater velocity, pump and pipeline characteristic, and the effectiveness of air valves along the main (Carne *et al.* 1982).

The adverse effects of gas locking include (USEPA, 1974; Boon *et al.*, 1977) :

- (i) an increase in the static head of the system and an increased friction losses causing a reduction in pumping rate and an increase in power consumption,
- (ii) pipe-line restriction and hence reduction in the capacity of the main,
- (iii) prevention of the operation of siphons in the main, which may initiate water hammer with a risk of fracture of the main, and
- (iv) odour nuisance when residual gases are released to the atmosphere.

The various measures taken to minimise gas locking include:

- (i) multiple injection points and release of residual gases at high points. Residual gases can be vented at high points by providing air release valves. In those cases where a very high stand pipe is required, the use of a return pipe taking a continuous bleed of wastewater and air from the high point to a suitable point in the wastewater collection system has been recommended (USEPA, 1974),
- (iii) maintaining the velocity of wastewater above 0.6 m/s and a Reynold's Number of at least 300,000 to ensure that undissolved gases remain entrained in the wastewater (Newcombe *et al.*, 1979),
- (iv) using high efficiency dissolvers, and
- (v) injecting air or oxygen after down grades.

### 2.7.2. pH change

The injection of air or oxygen in pressure mains (which is essentially a closed reactor) to achieve high concentrations of dissolved oxygen, may cause a decrease in the wastewater pH due to an accumulation of CO<sub>2</sub> (Vivona, 1980). According to Pomeroy and Lofy (1977) it is unlikely that this effect would cause any significant problem .

## 2.8. COSTS OF IN-SEWER AERATION AND OXYGENATION

The total cost of air injection consists of capital recovery cost, equipment maintenance cost, and the cost of energy for air compression and drying (Vivona, 1982; Speece *et al.*, 1990). Electricity, and hence the energy costs, vary widely depending upon the location and demand. Typical energy efficiency of force main aeration has been reported to be about 0.3 to 1 kg of O<sub>2</sub> per kw-h of electrical energy (Pomeroy and Lofy, 1977) and it has been shown that the cost of air systems increases exponentially with an increase in the DO level (Speece *et al.*, 1990).

As compared with air which is free, the cost of oxygen injection also includes the cost of oxygen generation and varies with the location of the site and the demand. Oxygen can be either delivered to the site in liquid form or can be generated on-site using the Pressure Swing Absorption System for higher demands (USEPA, 1985).

In the UK, typical on-site generation costs of oxygen (production and dissolution) supplied by BOC varies from £20 to £45 per tonne (BOC, 1994). The equivalent cost for cryogenic oxygen delivered at site as a liquid was £37 to £80 per tonne varying as a function of size of the demand, transportation distance, and availability for delivery. The oxygen transfer power



has been claimed up to 10 kg O<sub>2</sub>/kw-h. The cost of oxygen may be higher outside the UK, especially where the technology is not so well developed. Typical costs (budget level estimates: +30%, -15%) for direct air injection and direct oxygen injection in the USA are given in Table 2.4 and 2.5 respectively. If separate dissolution system is required, then this cost (Tables 2.4 and 2.5) would increase depending on the cost of the dissolution system. Table 2.6 shows typical costs of some of the oxygen dissolution systems marketed by CIG, Australia.

**Table 2.4-Typical costs for direct compressed air injection into a force main for H<sub>2</sub>S control (USEPA, 1985)**

Condition	Capital cost *( 1984 US \$)
Flow = 37, 850 m <sup>3</sup> /day	
Pipe: diameter = 25.4 cm	
Length = 1, 600 m	21, 000
Pressure = 158 kPa	
Air flow = 2.5 m <sup>3</sup> /min	
Flow = 37, 850 m <sup>3</sup> /day	
Pipe: diameter = 61 cm	
length = 1, 600 m	54, 000
Pressure = 158 kPa	
Air flow = 15.3 m <sup>3</sup> /min	
Note: *includes concrete pad, compressor, piping, valves, start-stop, controls, pre-fab buildings and installation.	

**Table 2.5-Typical costs for direct oxygen injection into a force main for H<sub>2</sub>S control (US EPA, 1985)**

Condition	Capital cost *( 1984 US \$)	
	Capital cost (\$)	Oxygen (\$/year)
Flow = 37, 850 m <sup>3</sup> /day		
Pipe: diameter = 36 cm	20, 000	9, 000 <sup>a</sup>
length = 1, 600 m		
O <sub>2</sub> required = 83 kg/day		
Flow = 37, 850 m <sup>3</sup> /day		
Pipe: diameter = 76 cm	50, 000	20, 000 <sup>b</sup>
length = 1, 600 m		
O <sub>2</sub> required = 310 kg/day		

Notes: \*includes concrete pads, liquid O<sub>2</sub> storage vessel, vaporiser, piping, start-stop controls and installation.

<sup>a</sup> Based on O<sub>2</sub> cost of \$0.31/kg

<sup>b</sup> Based on O<sub>2</sub> cost of \$0.18/kg

**Table 2.6-Costs of various oxygen dissolving techniques (Ogston, 1994)**

Dissolver Type	Capital Cost (1994 Aust. \$)
Direct Injection Double Injection Assembly/ dry well pump station	4332
Double Injection Assembly/wet well pump station	3800
Sidestream Venturi : Type D inclu. pump and nominal pipe work	37400
Single Stage U-Tube : 300 mm	17500
400 mm	25900
500 mm	36000
900 mm	90000
3-Stage U-Tube : 300 mm	33000
400 mm	44200
500 mm	66000
900 mm	180000
In-Line Nozzle : 300/100	6000
500/300	7300

The selection of air or oxygen for a particular system would depend upon a number of factors including the objective of DO enhancement, total oxygen demand, location of site, power costs, and characteristics of the sewerage system. The final decision regarding the use of air or oxygen system would be based on a detailed technical and economical evaluation of both systems and safety considerations. If the objective is to control sulphides only, then in many situations the overall cost of an air system may be less than that of an oxygen system. The use of oxygen however, would be economically attractive, if the additional benefits of oxygen addition, i.e. in-sewer treatment, is considered (Pomeroy and Lofy, 1977; Mozell, 1985; Speece *et al.*, 1990). Considerable savings in investment have been reported when oxygen injection was used for sulphide control and in-sewer treatment (Newcombe *et al.*, 1979; Hutchinson, 1981).

## 2.9. SUMMARY

The following main conclusions can be drawn from this literature survey on in-sewer treatment:

- (i) by maintaining aerobic conditions in sewers, a significant degree of in-sewer treatment could be achieved,
- (ii) the in-sewer purification could be enhanced either by seeding the sewers by the addition of activated sludge or by increasing the slime supporting area. The former method however, would be favoured, because of the operational problems and high cost associated with the latter,
- (iii) the oxygen requirements of flowing wastewater within a sewer vary greatly depending upon the age and temperature of wastewater, concentration of biomass, concentration of organic matter, and the wetted area. Although shallow gravity sewers may maintain

themselves aerobic, anaerobic conditions would prevail in pressure mains and deep gravity sewers, unless the dissolved oxygen is supplemented artificially,

- (iv) both air and oxygen injection are widely used for the control of sulphide in pressure mains. In-sewer oxygen injection techniques for both pressure mains and gravity sewers are well developed, and
- (v) the field of in-sewer treatment has been investigated to a limited extent. Also, due to limited operating experience, a thorough understanding of the process and hence, detailed design criteria of in-sewer treatment system have not been developed. A great deal of research and operating experience would be required before detailed design criteria for such a system could be developed.

## AIMS OF RESEARCH

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The review of literature has shown that there exists a strong possibility of using urban sewerage systems as aerobic biological treatment facilities for reducing the load on existing overloaded wastewater treatment plants or reducing the size of a proposed plant. Under favourable conditions i.e. where the retention time is long and the temperature is high (Hemmings *et al.*, 1983), or where the wastewater is seeded with activated sludge (Green *et al.*, 1985), the degree of treatment which could be achieved in sewers may be comparable to that found in a conventional secondary treatment plant.

Despite the installation of full-scale in-sewer treatment systems in Bath, UK, and in Boulder Bay in Australia, there have been few scientific investigations on in-sewer treatment. Most of the studies undertaken so far either involved direct *in-situ* measurements of BOD<sub>5</sub>, or were focused merely on the removal of organic matter in a particular system which had a specified retention time. No study appears to have been undertaken to fully investigate the treatment potential of sewers while taking into account various operational parameters of an in-sewer treatment system. Furthermore, many studies seem to have been undertaken without any reference to previous investigations resulting in the field of in-sewer treatment remaining in its infancy.

Consequently, to gain a better understanding of in-sewer treatment systems, there is a need to explore, in-depth, the treatment potential of sewers and to identify and optimise the various parameters affecting in-sewer treatment.

This study, therefore, will be undertaken with the broad objectives of investigating the possibility of using urban sewerage systems for the treatment of domestic wastewater during its transit. The particular objectives of this study are:

- (i) to investigate the degree of in-sewer treatment which may be achieved by the suspended biomass normally present in the wastewater while aerobic conditions are maintained in the sewer,
- (ii) to study the possibility of enhancing in-sewer aerobic treatment by increasing the suspended biomass of the flowing wastewater by the addition of activated sludge at the inlet of the sewer,
- (iii) to study the oxygen requirements of various treatment alternatives stated under objectives (i) and (ii), and
- (iv) to study the settling characteristics of the effluent from the treatment systems stated under objective (i) and (ii).

## CHAPTER 4

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### EXPERIMENTAL DESIGN, MATERIALS AND METHODS

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#### 4.1. INTRODUCTION

To achieve the research objectives, the current study was divided into two phases. The first phase was undertaken in the Environmental Engineering Laboratory, University of Newcastle upon Tyne, while the second phase was undertaken at the Central Laboratories of the Water Authority of Jordan, Amman, Jordan. A description of the experimental design, materials and experimental methods used in both phases is given in the following sections.

#### 4.2. EXPERIMENTAL DESIGN

##### 4.2.1. Phase-I studies

In the first phase, which comprised three parts, a general study of the possibility of using sewers as a treatment facility was investigated. The work undertaken in each part of the Phase-I is as follows:

##### (i) Part-1: Preliminary studies

- (a) a comparative study of aerobic and anaerobic conditions in sewers on in-sewer removal of COD, and SS at the downstream wastewater treatment plant,
- (b) a comparative study of the effect of the wastewater velocity (mixing speed) in the sewer on in-sewer removal of soluble COD, and SS at the downstream wastewater treatment plant, and

- (c) a comparative study of the effect of the dissolved oxygen source (air or oxygen) on in-sewer removal of soluble COD, and SS at the downstream wastewater treatment plant.

**(ii) Part-II: Addition of oxygen to raw wastewater**

- (a) the removal of organic matter in sewer under aerobic conditions while taking into account the effect of variations in the wastewater strength, and
- (b) the effect of temperature variations on in-sewer removal of soluble COD.

**(iii) Part-III: Addition of oxygen to seeded wastewater**

In this part of the study, the possibility of enhancing in-sewer aerobic treatment by the addition of activated sludge, at an initial concentration of 100-1000 mg VSS/l, at the inlet of a sewer, was investigated.

**4.2.2. Phase-II studies**

In this phase of the current study, a case-study was undertaken to assess:

- (i) the degree of treatment that could be achieved in the Greater Amman Siphon (GAS) under aerobic conditions without any seeding, and
- (iii) the degree of treatment that can be achieved in the GAS under aerobic conditions with the addition of activated sludge, at an initial concentration of 100-500 mg VSS/l, at the inlet.



### **4.3. MATERIALS AND METHODS**

#### **4.3.1. Phase-I studies**

##### **(i) Feed wastewater**

The raw wastewater (screened) used in this study was collected from the inlets to three local wastewater treatment plants i.e. Durham, Sacriston, and Broohaugh, all treating mainly domestic wastewater. Because of the length of the time required to carry out a batch experiment, it was not possible to use the wastewater immediately after collection. The grab samples of the wastewater were therefore stored at 4°C just after arrival in the laboratory and were used for batch tests within 24 hours of collection.

##### **(ii) Seed sludge**

The activated sludge used for seeding the raw wastewater was the returned sludge collected from Durham Wastewater Treatment Plant. The returned activated sludge (RAS) was allowed to settle for one hour just after arrival in the laboratory and the supernatant decanted in order to increase the suspended solids concentration. The sludge was aerated overnight before being used. As the sludge was collected from a wastewater treatment plant treating mainly (98%) domestic wastewater, the effect of acclimatisation was considered to be negligible.

Before the start of an experiment, the suspended solids (SS) and volatile suspended solids (VSS) of the returned sludge were determined and the volume of RAS to be added to the wastewater to achieve a given concentration was calculated by the mass balance equation as follows:

$$(V_w + V_a) \times VSS_r = V_w \times (VSS_w) + V_a \times (VSS_a) \quad (4.1)$$

Where:

$V_w$  = volume of the wastewater in the batch reactor (litres),

$V_a$  = volume of returned activated sludge (litres) to be added to  $V_w$  to achieve an active biomass concentration of  $VSS_r$  (mg/l) in the batch reactor,

$VSS_w$  = volatile suspended solids of the raw wastewater (mg/l), and

$VSS_a$  = volatile suspended solids of returned activated sludge (mg/l).

It should be noted that in Equation 4.1 the value of the VSS of the raw wastewater prior to the addition of sludge ( $VSS_w$ ) was assumed to be zero.

### (iii) Reactors

A sewer conforms perfectly to plug flow conditions (Hemmings *et al.*, 1983). The transport of the wastewater in a sewer, therefore, could be simulated by a batch reactor with a retention time equivalent to that of the retention time of the wastewater in the sewer (Green *et al.*, 1985). The retention time in a sewer is related to its length as follows:

$$t = \frac{L}{V} \quad (4.2)$$

Where:

$t$  = retention time in the sewer (hours),

$L$  = length of the sewer (m), and

$V$  = wastewater velocity (m/s).

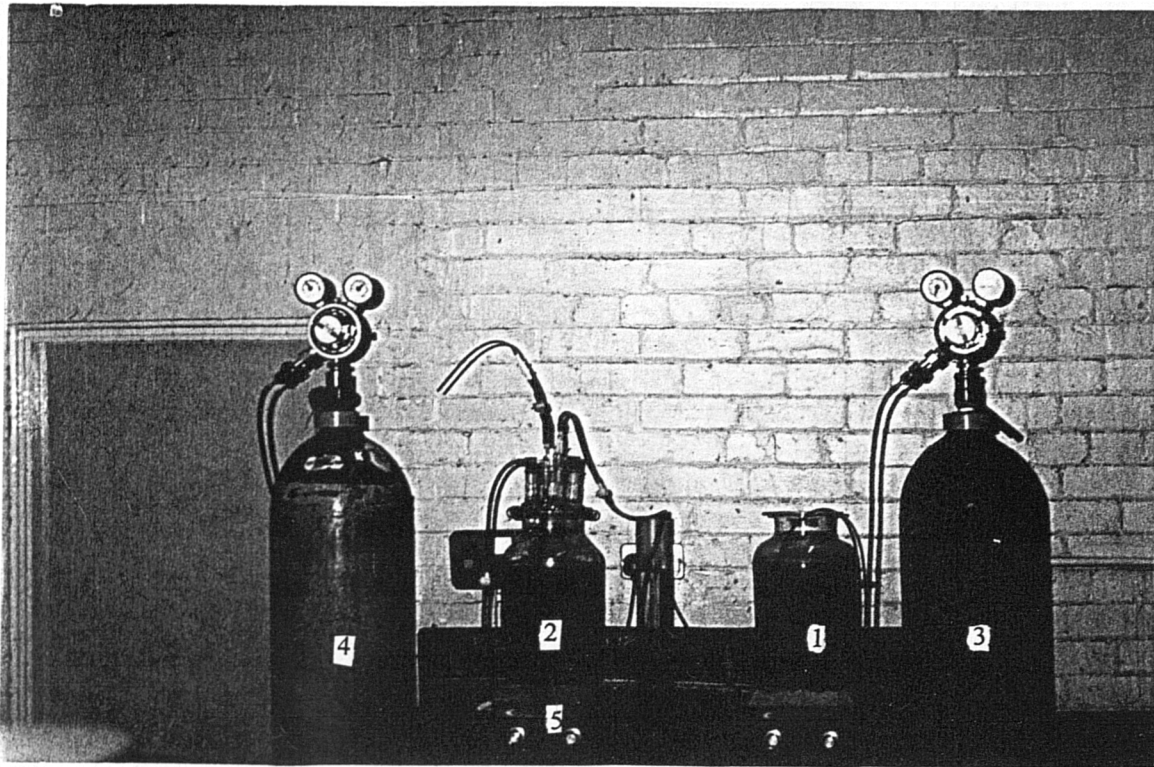
In the present study two different types of batch reactors were used to simulate the transport of the wastewater in a linear gravity sewer. The 5-litre magnetic-stirred glass reactors were used to compare the aerobic and anaerobic gravity transport in sewers (Section 5.2) while in order to have a greater sample volume, maintain a constant temperature and achieve better mixing, especially at high seed concentrations, it was decided to use the impeller mixed reactors for the rest of experimental work. A brief description of both types of reactor is given in the following sections.

#### **(a) Batch reactors employing magnetic stirrer**

These reactors (Figure 4.1) consisted of glass aspirators each having a working volume of 5 litres. The anaerobic reactor was kept air tight and was injected with nitrogen at the start of the experiment to exclude the entrained air. In the aerobic reactor, the dissolved oxygen (DO) was maintained well above 2 mg/l by intermittent injection of oxygen through a porous diffuser placed at the bottom of the reactor. Both the reactors were mixed using a magnetic stirrer and maintained at ambient temperature (average 22°C).

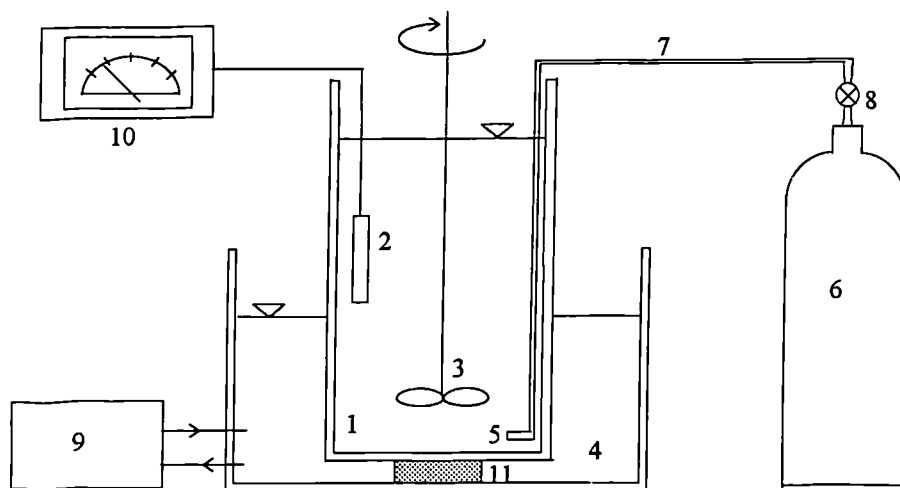
#### **(b) Impeller mixed batch reactor**

This reactor consisted of a 20-litre cylinder which had a working volume of 10-15 litres. The reactor contents were mixed using a variable speed mechanical stirrer. All the experiments except those designed to investigate the effect of mixing speed on in-sewer treatment (Section 5.3) were undertaken at a mixing speed of 100 rpm. A constant temperature was maintained by placing the reactor in a controlled-temperature water bath. Aerobic conditions were maintained in the reactor by intermittent supply of oxygen through four porous diffusers placed at the bottom of the reactor. A schematic drawing of the experimental set-up for impeller mixed batch reactor is shown in Figure 4.2



**Figure 4.1. Pictorial view of magnetic mixed batch reactors**

1. Aerobic reactor; 2. Anaerobic reactor; 3. Oxygen cylinder; 4. Nitrogen cylinder;  
5. Magnetic mixer.



**Figure 4.2- Experimental set-up for impeller-mixed batch reactors**

1. Batch reactor; 2. DO probe; 3. Impeller; 4. Water bath; 5. Diffuser; 6. Oxygen cylinder;  
7. Oxygen supply line. 8. Valve; 9. Temperature controller; 10. DO meter; 11. Support slab

**(iv) Reactor operation and feeding**

The containers of wastewater were held in a water bath before feeding to equilibrate with the operating temperature. After the wastewater had reached the desired temperature, it was fed to the batch reactor manually, the reactor contents were mixed and an influent sample was taken. The oxygen supply was then switched on and effluent samples were taken at specified intervals of time. For tests involving the addition of active biomass to the wastewater (Chapters 7 and 8), a specified quantity of the RAS (Equation 4.1) was added to the reactor just before the oxygen supply was turned on. The reactor contents were emptied and disposed of at the end of each individual test. The reactor was then washed thoroughly with water to ensure that no biomass growth occurred on the walls of the reactor.

**(v) Measurement of dissolved oxygen**

The dissolved oxygen (DO) in the reactor was measured using a dissolved oxygen probe (YSI, Model 57). The DO meter was calibrated at the start of each experiment by reading the value shown on the meter against a sample of known DO concentration (determined by Azide modification of Winkler method), and was zeroed by placing the probe in a solution of sodium sulphite and cobalt chloride.

**(vi) Analytical methods**

Influent and effluent samples were analysed immediately or stored at 4°C. All the analysis were carried out within 24 hours. Unfiltered samples were used for the determination of total COD and suspended solids (SS) while filtered samples (using Whatman GF/A) were used for the determination of soluble COD and BOD<sub>5</sub>. All analyses were carried out

according to *Standard Methods* (1989). A description of various tests is given in Table 4.1.

**Table 4.1-Analytical methods and sensors or meters used** (*Standard Methods*, 1989)

Parameter	Method
BOD <sub>5</sub> (ATU)	Dilution method, DO by modified Winkler method
DO	Electrode
Soluble COD	Closed reflux, titrimetric Method
SOUR	See text
SS	Total suspended solids dried at 103-105°C
SVI	Static method
Total COD	Open reflux, titrimetric method
VSS	Fixed and volatile solids ignited at 550°

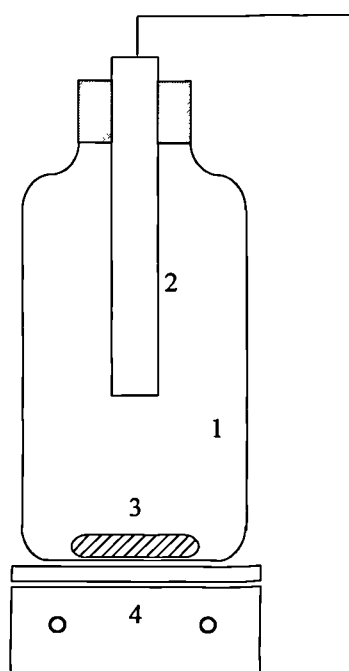
#### **(vii) Measurement of settlability**

A one-litre glass cylinder (Appendix A) was used as a settling tank. For experiments involving the addition of activated sludge to the wastewater, the settleability was measured using Sludge Volume Index (*Standard Methods*, 1989).

#### **(viii) Measurement of oxygen uptake rate of suspended biomass**

The oxygen uptake rate of suspended biomass (SOUR) was measured using an SOUR reactor consisting of a one-litre air tight plastic container which was fitted with a dissolved oxygen meter (Figure 4.3). The contents of the SOUR reactor were kept mixed

by a magnetic stirrer. At specified intervals of time, the SOUR reactor was filled with wastewater or mixed liquor from the main batch reactor and the DO level increased to 6-8 mg/l if required. The change in the DO level in the reactor over time was monitored with a DO probe. The contents of the SOUR reactor were transferred to the batch reactor afterwards. The gradient of DO concentration was taken as SOUR (mg/l.h).



**Figure 4.3. The SOUR reactor**

1. Plastic container; 2. DO probe; 3. Stir bar; 4. Magnetic stirrer

#### **(ix) Calculation of removal efficiencies**

The soluble COD or BOD<sub>5</sub> removal efficiencies for a given retention time were calculated on the basis of the soluble COD or BOD<sub>5</sub> of the wastewater at the start of the experiment and after that retention time, without settling the effluent. A similar method was used for the calculation of total COD removal efficiencies. The removal efficiency R is given by:

$$R = 100\left(\frac{S_0 - S_t}{S_0}\right) \quad (4.3)$$

Where:

R = COD or BOD<sub>5</sub> removal efficiency (%) after t hours from the start of the experiment,

S<sub>0</sub> = COD or BOD<sub>5</sub> concentration (mg/l) at the start of the experiment, and

S<sub>t</sub> = COD or BOD<sub>5</sub> concentration (mg/l) after t hours from the start of the experiment.

The amount of soluble COD removal (mg/l) over a given retention time (t hours) was found as follows:

$$\text{RSCOD} = (S_0 - S_t) \quad (4.4)$$

Where: RSCOD = soluble COD removal (mg/l) after a given retention time t (hours).

For experiments involving the addition of activated sludge to the wastewater, the removal efficiency was calculated using Equation 4.3, except the influent soluble CODs and soluble BOD<sub>5</sub>s were calculated by applying mass balance to the CODs and BOD<sub>5</sub>s of the raw wastewater and the activated sludge as follows:

$$S'_0 = \frac{S_a V_a + S_0 V_w}{V_w + V_a} \quad (4.5)$$

Where:

S'<sub>0</sub> = influent SCOD or SBOD<sub>5</sub> for seeded experiments (mg/l),

S<sub>a</sub> = SCOD or SBOD<sub>5</sub> (mg/l) of the activated sludge at the start of the experiment ,



$V_a$  = Volume of returned activated sludge (litres) added to the wastewater, and

$V_w$  = volume of the wastewater in the batch reactor (litres).

It must be noted that terms “removal efficiencies” or “treatment efficiencies” used throughout this dissertation refer to the treatment efficiencies achieved by the suspended biomass only, as the effect of slime layer on the sewer wall was not considered.

### **4.3.2. Phase-II studies**

#### **(i) Description of the Greater Amman Siphon**

The Greater Amman Siphon (GAS) conveys the screened and degrittled wastewater and septage, from the city of Amman from Ain Ghazal pre-treatment facility, to Al-Samra wastewater stabilisation ponds system (AWSP). The GAS (Appendix A-1) is a pressure gravity pipe line and consists of 52 grade steel with welded joints. It is 38.6 km long and has a diameter of 1228 mm. Its elevation at the inlet near Ain Ghazal is 688 m while at the discharge point the elevation is 580 m. The elevation at the lowest point of the GAS, which is at a distance of about 26 km from the inlet, is 460 m. The design pressure at this point is approximately 23 bars. The retention time in the GAS averages 8 hours.

The total flow at the inlet of the GAS averages 115,000 m<sup>3</sup>/day, with average domestic wastewater and the septage flows amounting to 110,000 m<sup>3</sup>/day and 5000 m<sup>3</sup>/day, respectively. A step flow of about 30,000 m<sup>3</sup>/day is pumped into the siphon at Zarqa, at a distance of about 25 km from the head of the siphon. For the purpose of the current study however, the step-flow to the GAS was ignored, as a separate WWTP is being planned for Zarqa area.

**(ii) The Al-Samra wastewater stabilisation ponds system**

The Al-Samra wastewater stabilisation ponds system, the largest WSP system in the Middle East, consists of three trains, each comprising of anaerobic, facultative, and maturation ponds. Salient features of the AWSP system are given in Table 4.2. The AWSP system in addition to treating and disposing the major portion of domestic wastewater, septage and industrial wastewater from the city of Amman, also handles and treats the wastewater from Zarqa, Ruseifa and Hashimiya.

**Table 4.2-Salient features of AWSP system (Al-Salem, 1987)**

Effective area	181 hectares
Effective volume	2.8 million m <sup>3</sup>
Design flow-Average dry weather	68,000 m <sup>3</sup> /day
Peak wet weather	148,000 m <sup>3</sup> /day

The AWSP system, which was commissioned in 1985, was designed to treat an average dry weather flow of 68,000 m<sup>3</sup>/day. The system however, is currently overloaded by 150%, with the average daily flow amounting to 170,000 m<sup>3</sup>/day. The Water Authority of Jordan (WAJ) intends to upgrade the AWSP system and studies are underway to determine the various alternative options for extension.

**(iii) Feed wastewater**

The wastewater used in this study was collected from inlet to the GAS, at Ain Ghazal. The wastewater consisted of a mixture of domestic wastewater and septage, both of which were separately pre-treated at Ain Ghazal pre-treatment works comprising of screens and

aerated grit chambers. The grab samples of the wastewater were collected at Ain Ghazal between 8 and 9 am and were subjected to batch testes at WAJ laboratories at Wadi Seer, within three hours of collection.

#### **(iv) seed sludge**

The returned activated sludge used for seeding the raw wastewater was collected from Abu Nusair extended aeration activated sludge plant. The quantity of sludge added to the raw wastewater to attain a given activated sludge concentration in the reactor was calculated using Equation 4.1.

#### **(v) Experimental methodology**

The impeller-mixed batch reactors used in this study (Appendix A-2) were similar to those described in Sections 4.3.1, except the temperature of the wastewater was maintained at ambient. The experimental methods used for this study were essentially the same as described in Sections 4.3.1, except for some of analytical methods as described in the following section.

#### **(vi) Analytical methods**

Filtered samples (using Whatman GF/C) were used for the determination of soluble COD and BOD<sub>5</sub>. As practised in WAJ laboratories, no chemical (i.e. ATU) was added to the dilution water to arrest the nitrification during the BOD<sub>5</sub> test. All analyses were carried out according to *Standard Methods* (1989).

## CHAPTER 5

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### PRELIMINARY STUDIES

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#### 5.1. INTRODUCTION

A sewer constitutes a dual phase system wherein the heterotrophic bacteria are present in suspended form in the flowing wastewater and in attached form in the slime layer on the walls of the sewer.

During the transit of wastewater to treatment plants, the dissolved oxygen in the wastewater is depleted as a result of the respiration of the indigenous biomass. Although shallow gravity sewers may maintain themselves aerobic, anaerobic conditions prevail in deep gravity sewers and pressure mains unless the dissolved oxygen is supplemented (USEPA, 1974). The time for the onset of anaerobic conditions could be as short as 10 minutes in small pipes, and 20 to 30 minutes in large pipes (Pomeroy, 1992).

Although a few studies have been undertaken in the past to investigate the removal of organic matter in sewers under aerobic conditions (Section 2.2), there does not appear to be much information in the literature regarding the effect of anaerobic conditions in sewers on the removal of organic matter (COD or BOD) and the subsequent settleability at the downstream treatment works.

The main objective of the work described in this chapter was to compare the removal of soluble organic matter in the wastewater and the subsequent primary settleability, under aerobic and anaerobic conditions in sewers. In addition to this, an investigation was carried out on the effect of turbulence (wastewater velocity), and the dissolved oxygen source (air

or oxygen) on the removal of soluble COD during the simulated aerobic gravity transport, and the subsequent primary settleability, in order to establish the optimum operating conditions for further experimental work.

## 5.2. AEROBIC VERSUS ANAEROBIC CONDITIONS IN SEWERS

To compare the aerobic and anaerobic transport of domestic wastewater in a gravity sewer, two identical 5-litre batch reactors (Figure 4.1) were run in parallel at ambient temperature (average 22°C). A detailed description of the experimental methodology used for this study is presented in Section 4.3.1.

### 5.2.1. Soluble COD removal

Table 5.1 summarises the average SCOD removal efficiencies under different redox conditions. The variation in SCODs of the individual wastewaters over a retention period of 8 hours is illustrated in Figures 5.1 and 5.2.

**Table 5.1-Soluble COD removal efficiencies under different redox conditions**

Retention time	Redox condition	SCOD removal (%)
2	Anaerobic	12±4*
	Aerobic	15±3
4	Anaerobic	13±4
	Aerobic	25±3
6	Anaerobic	11±6
	Aerobic	35±3
8	Anaerobic	6±7
	Aerobic	36±6

Notes (i) \* average±SD (iii) average influent SCOD=291±33 mg/l  
(ii) average influent SS=366±75 mg/l (iv) average temperature= 22°C

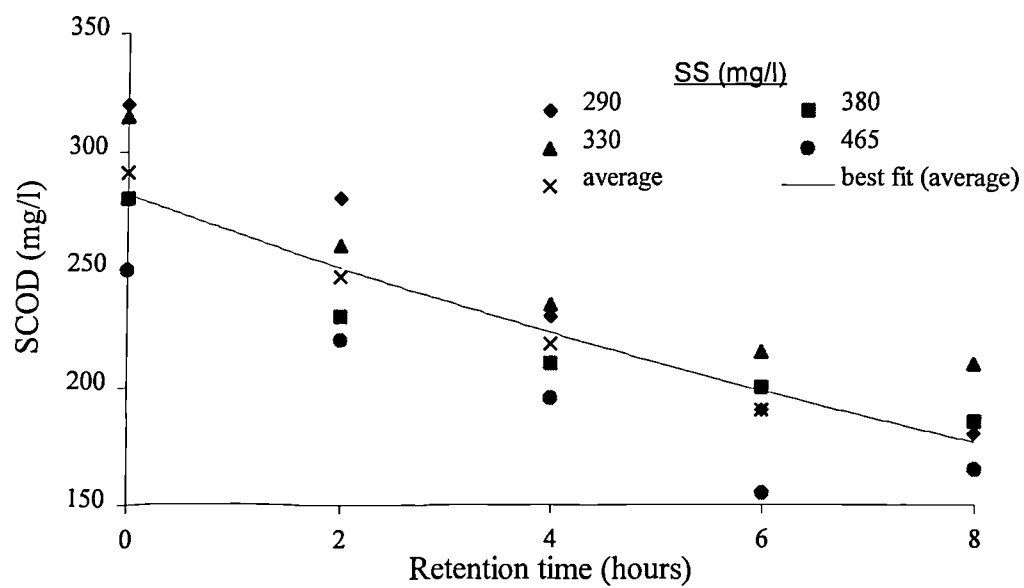


Figure 5.1-Variation in SCOD of domestic wastewater under aerobic conditions.

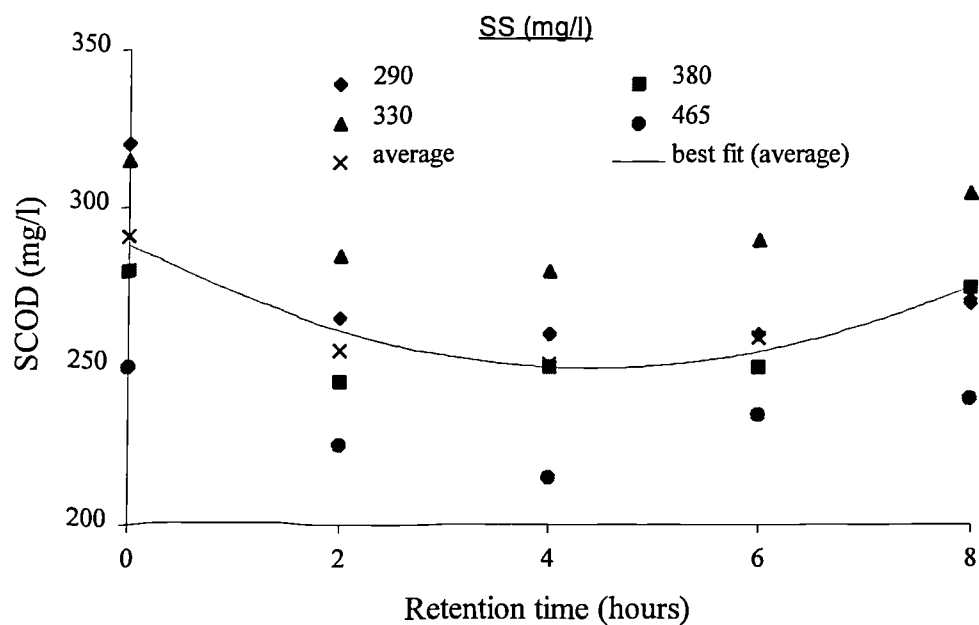


Figure 5.2-Variation in SCOD of domestic wastewater under anaerobic conditions.

As can be seen from Figure 5.1, the SCOD in the wastewater under aerobic conditions showed a gradual decrease with the passage of time. The SCOD removal over 8 hours ranged from 80-140 mg/l. The net SCOD removal rate averaged 13 mg/l.h over a retention period of 8 hours. The SCOD removal rate was at a maximum during the first two hours of oxygen addition and showed a decrease with increase in retention time.

Under anaerobic conditions, the SCOD in the reactor showed a decrease initially which was then followed by an increase (Figure 5.2). The net SCOD removal ranged from 30-35 mg/l over the first four hours from the start of the experiment. During the next four hours, the SCOD in the reactor increased by an average of 25 mg/l (n=3). The net SCOD removal over 8 hours thus decreased to 5-10 mg/l.

### **5.2.2. Total COD removal**

The influent total COD (TCOD) of the wastewater averaged 769 mg/l (SD=55). The TCOD removal efficiency as calculated on the basis of the TCOD of unsettled effluent (Equation 4.3) after a retention period of 8 hours averaged 8% (SD=8) and 11% (SD=6) under aerobic and anaerobic conditions, respectively.

### **5.2.3. Effect of redox conditions on the settleability of solids**

To study the effect of a range of redox conditions on the settleability of SS downstream of the sewer, effluent samples taken from batch reactors after a retention time of 6 hours were settled in a 1 litre cylinder (Section 4.3.1) for 1 hour and the SS of the supernatant was determined. The results of the settleability tests are shown in Table 5.2.

**Table 5.2-Effect of redox conditions on the removal of SS**

Redox condition	Effluent SS	Supernatant SS	Solids removal
—	mg/l	mg/l	%
Anaerobic	335±48*	145±18	56±6
Aerobic	380±76	105±9	72±4

Notes: (i) \* average ± SD

(ii) Influent SS = 370±75 mg/l

(iii) n = 4

In the aerobic reactor, the raw wastewater appeared to assume the form of a dilute suspension of activated sludge after the first hour of oxygen addition. The net changes in SS of the raw wastewater over a retention period of 6 hours varied from 0 to +6% (average=+3, SD=2) under aerobic conditions, and 0 to -18% (average -7%, SD=8) under anaerobic conditions. A comparison of the settleability results for both redox conditions (Table 5.2) showed that the average solids removal under aerobic conditions was 29% higher than that under anaerobic condition.

### 5.3. EFFECT OF TURBULENCE ON IN-SEWER AEROBIC TREATMENT

Velocities in sewers vary greatly due to the variation in wastewater flow. To study whether these changes in velocity (turbulence) would have any effect on the removal of soluble organic matter and primary settleability under aerobic conditions, and subsequently to select an appropriate mixing speed to be used for further experimental work, four identical impeller-mixed batch reactors (Figure 4.2), were run in parallel at different mixing speeds, at a constant temperature (20°C). Three different wastewaters were subjected to batch tests at mixing speeds of 50, 100, 200 and 300 rpm. Effluent samples, taken after a retention time of 6 hours were settled in a 1 litre cylinder (Section 4.3.1) for 1 hour and the SS of the



supernatant was determined, while those taken after a retention period of 8 hours were analysed for SCOD. Table 5.3 summarises the average SCOD and SS removal efficiencies at different mixing speeds.

**Table 5.3- Effect of mixing speed on SCOD and SS removal under aerobic conditions**

Mixing speed (rpm)	50	100	200	300
SCOD removal (%)	41±4	42±4	39±3	42±5
Solids removal (%)	74±12	77±10	74±10	47±3

Notes: (i) average influent SCOD = 252±113 mg/l

(ii) average influent SS = 437±163 mg/l

(iii) n = 3

To determine statistically whether the changes in mixing speed had significantly affected the SCOD removal, one-way analysis of variance (ANOVA) tests (Metcalf, 1994) using Minitab 9.2, were carried out on the raw data. The results of the ANOVA tests (Appendix B-1) indicated that at 5% significance level, there was no significant difference ( $p > 0.05$ ) in terms of SCOD removal, due to changes in mixing speed in the range of 50-300 rpm, over a retention period of 8 hours. Similarly, no significance difference was noted in SS removal efficiencies at mixing speeds of 50-200 rpm. An increase in mixing speed beyond 200 rpm however, was found to significantly affect the amount of SS removed (Table 5.3 and Appendix B-1) possibly due to increased turbulence in the batch reactor.

#### 5.4. AIR VERSUS OXYGEN FOR IN-SEWER AEROBIC TREATMENT

Both air and oxygen are widely used for sulphide control in pressure mains, while in-sewer oxygenation for gravity sewers has also been reported (Section 2.6). In-sewer oxygenation, as compared to in-sewer aeration, has been claimed to result in an improved primary

settleability (Newcombe *et al.*, 1979; Mozell, 1985). To compare the removal of soluble organic matter and primary settleability for in-sewer oxygenation and in-sewer aeration systems, two identical impeller mixed batch reactors (Figure 4.2), one using pure oxygen and other air as a source of dissolved oxygen, were operated in parallel at 20°C. Effluent samples, taken after a retention period of 6 hours were settled in a 1 litre cylinder for 1 hour and the SS of the supernatant was determined, while those taken after a retention period of 8 hours were analysed for SCOD. Table 5.4 summarises the average SCOD and SS removal efficiencies for air and oxygen systems.

**Table 5.4-Effect of dissolved oxygen source on SCOD and SS removal**

Dissolved oxygen source	Air	Oxygen
SCOD removal (%)	36±5	39±5
SS removal (%)	66±4	70±4

Notes: (i) average influent SCOD = 286±28 mg/l  
(ii) average influent SS = 308±33 mg/l  
(iii) n = 4

From the results shown in Table 5.4, it appears that the use of air when compared with oxygen resulted in 3 and 4% higher SCOD and SS removal, respectively. To assess statistically the effect of dissolved oxygen source on SCOD and SS removal however, one-way ANOVA test was applied to the raw data collected during this study. The results of the ANOVA tests (Appendix B-2) revealed that at a significance level of 5%, there was no significant difference ( $p > 0.05$ ) in soluble COD removal, for both air and oxygen systems, over a retention period of 8 hours. A similar effect was found for the suspended solids removal efficiencies after a retention period of 6 hours (Appendix B-2).

## 5.5. DISCUSSION

The results shown in Table 5.1 and Figures 5.1 and 5.2, demonstrate that the greatest removal of soluble organic matter would occur under aerobic conditions. The higher removal rate during the first two hours under aerobic conditions is possibly due to oxidation of the readily biodegradable fraction of soluble organic matter while a subsequent decrease in the soluble COD removal rate may be attributed to an increase in soluble COD due to hydrolysis of the particulate organic matter (Section 6.7.1).

The results of the present study indicate that under anaerobic conditions, the soluble COD of the wastewater, in general, decreases during the first four hours which is then followed by an increase in SCOD with the passage of time, thus offsetting the early decrease. An increase in soluble COD of the wastewater during the latter stages of the experiments could be due to hydrolysis of particulate organic matter.

When the results of one experiment not conforming to the general trend (SS = 465 mg/l, Figure 5.1) were omitted, the SCOD removal over 8 hours under aerobic conditions averaged 95 mg/l. The corresponding SCOD removal under anaerobic conditions averaged 8 mg/l or one twelfth of that achieved under aerobic conditions. This is in reasonable agreement with the argument of Boon (as quoted by Pullin, 1977) that, under anaerobic conditions in sewers, the degradation rate of the wastewater would be about one fourteenth of that under aerobic conditions.

Greenfield *et al.* (1980) have reported a soluble BOD<sub>5</sub> removal pattern under anaerobic conditions similar to the SCOD removal pattern found in the present study. Nielsen *et al.*

(1992) reported that, under anaerobic conditions, the changes in dissolved protein of the wastewater were not significant over a retention period of 20 hours. Based on field studies involving a pressure main, Hvitved-Jacobson *et al.* (1995) concluded that, under anaerobic conditions, the reduction in soluble COD did not show any clear pattern. Kaijun *et al.* (1995), found that SCOD removal of the raw wastewater under anaerobic conditions varied widely during the hydrolysis phase. According to them, the main processes under such conditions during the first two weeks are hydrolysis and acidification.

The total COD results (Section 5.2.2) indicate that under aerobic conditions, the changes in total COD were significantly low as compared with soluble COD. This is in keeping with the argument of Newcombe *et al.* (1979) and Greenfield *et al.* (1980) that little changes are expected in total COD during aerobic transport in sewers because of the transformation of soluble and colloidal organic matter into additional biomass, which results in an increase in the total organic matter of the wastewater.

Results shown in Table 5.2 suggest that maintaining aerobic conditions in the sewers could result in a significant improvement in primary sedimentation, in addition to the removal of soluble organic matter. This observation is in accordance with the experience from the field investigations. Newcombe *et al.* (1979) and Hemmings *et al.* (1983) found that injection of oxygen in sewers resulted in significant reduction in load and improved primary sedimentation the downstream treatment works. According to Hutchinson (1981), the improvement in primary sedimentation as a result of oxygen addition is due to the fact that under aerobic conditions, biomass tends to flocculate as it is formed into rudimentary flocs similar to those found in activated sludge.

The overall results of the current study demonstrate that aerobic treatment, when compared with anaerobic treatment, would be the most promising method for in-sewer biological treatment, and could result in a significant removal of soluble organic matter and increased primary sedimentation. Also, in-sewer anaerobic treatment is less likely to be favoured due to potential odour and corrosion problems associated with anaerobic conditions.

The results of the present study indicate that changes in mixing speed (in the range of 50-200 rpm) would not significantly affect the soluble COD removal and the subsequent settleability. According to Pomeroy and Parkhurst (1972), the mixing speed does not make a measurable difference in oxygen uptake rate of wastewater, as long as it is sufficient to keep the organic solids suspended. Newcombe *et al.* (1979) also noted a similar affect.

The velocity of wastewater in sewers however, has been reported to influence strongly the dispersion of air or oxygen injected in the sewer (Newcombe *et al.* 1979; Speece *et al.*, 1990). For in-sewer oxygenation to be effective, and to prevent gas locking (Section 2.4), Newcombe *et al.* (1979) recommend a wastewater velocity of above 0.6 m/s and a Reynolds's Number of at least 300,000 to be maintained in the sewer.

From the results of the current study, in-sewer removal of soluble organic matter and the subsequent solids removal appear to be independent of the dissolved oxygen source (air or oxygen). In-sewer oxygenation however, may offer other advantages, as discussed in Section 2.6.

Based on the results of the preliminary studies, it was decided to focus further research work on the use of sewers as an aerobic treatment facility. A rotational speed of 100 rpm (corresponding to an average linear velocity of 1 m/s in the reactor) was selected to be used for the rest of the work, while in order to have a better operational control in the laboratory and avoid excessive turbulence, oxygen was chosen to be used as the source of dissolved oxygen.

## CHAPTER 6

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### ADDITION OF OXYGEN TO RAW WASTEWATER

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#### 6.1. INTRODUCTION

As emphasised in Chapter 5, heterotrophic micro-organisms are present in sewers in suspended form in the wastewater and in attached form in the slime layer on the wetted area of sewer walls. It has been indicated that, under aerobic conditions, these micro-organism would increase at a high rate thus resulting in a significant degree of in-line purification (Pomeroy and Parkhurst, 1972; Boon *et al.*, 1977).

This section describes the results of the extensive experimental work undertaken to investigate, in general, the degree of in-sewer treatment which could be achieved by the suspended heterotrophs normally present in the wastewater, under aerobic conditions.

To simulate the aerobic transport of wastewater in a linear gravity sewer, the raw domestic wastewater was maintained aerobic in a batch reactor (Figure 4.2) by the addition of oxygen, at atmospheric pressure. A detailed description of the experimental methodology adopted for this study is presented in Section 4.3.1.

#### 6.2. WASTEWATER CHARACTERISTIC

The raw (screened) wastewater used in this study was collected from inlets to three local wastewater treatment plants (WWTPs) namely Broomhaugh, Sacriston and Durham, all treating mainly domestic wastewater. Table 6.1 summarises the ranges of the characteristics of the wastewaters collected from the three WWTPs.

**Table 6.1-Characteristics (mg/l) of the wastewater used in simulation study**

WWTP	Broomhaugh	Sacriston	Durham
SCOD	70-215	255-390	250-400
SS	150-420	230-500	540-740
TCOD	280-750	540-1170	920-1480

As can be seen from Table 6.1, a considerable variation was noted in the strength of wastewater collected from the different WWTPs. The samples collected from Broomhaugh represented a wastewater having typically low soluble COD (SCOD) and low suspended solids (SS). Samples from Sacriston depicted a wastewater with typically high SCOD and low SS while those from Durham represented a wastewater with a high SCOD and high SS.

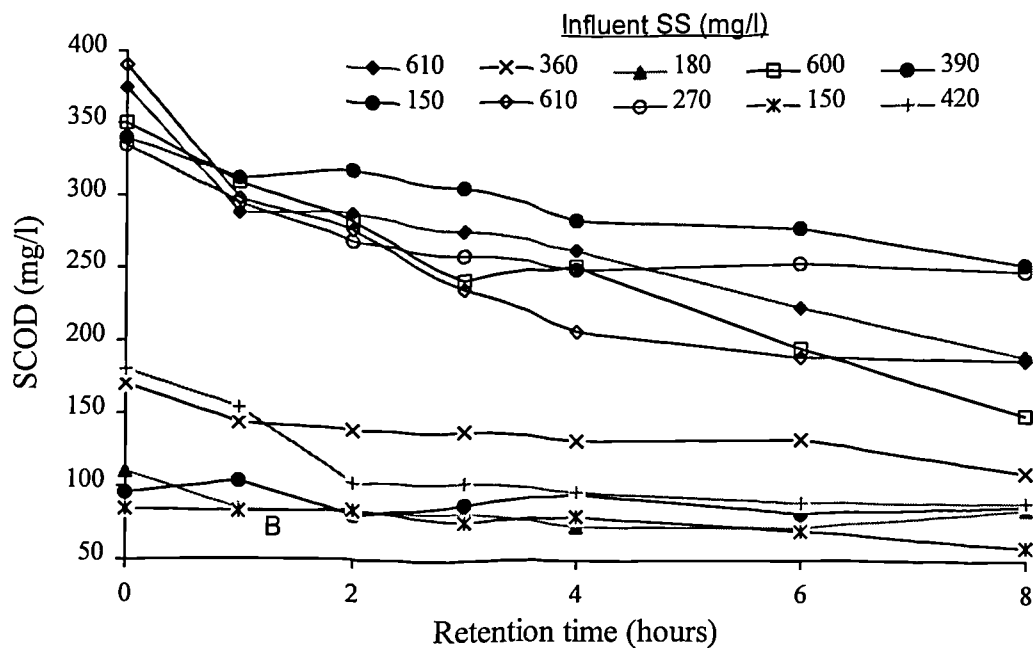
### 6.3. REMOVAL OF SOLUBLE ORGANIC MATTER

The removal of soluble organic matter from the wastewater was studied at three different temperatures i.e. 10°, 20° and 30°C. Initially a minimum of eight wastewaters were studied at each temperature. A wide variation was observed in the strength of the wastewater investigated at each temperature, and the removal of soluble organic matter in the wastewater was found to vary widely depending upon the initial suspended solids ( $SS_0$ ) and the initial soluble COD ( $SCOD_0$ ) of the wastewater. Consequently, it was decided to investigate more wastewaters at 20°C in order to have wide range of  $SS_0$  and  $SCOD_0$  concentrations, so that sufficient data for carrying out the statistical analysis to assess the effect of  $SS_0$  and  $SCOD_0$  on the SCOD removal, could be generated. SCOD was chosen to be the principal parameter for the measurement of organic matter in this study because of the large number of samples involved.



### 6.3.1. Removal of soluble organic matter at 10°C

Figure 6.1 shows the variation in the SCODs of the individual wastewaters over a retention period of 8 hours. Table 6.2 summarises the average SCOD removal efficiencies at 10°C while the corresponding average SCOD<sub>0</sub> and SS<sub>0</sub> concentrations are shown in Table 6.3.



**Figure 6.1-Variation in SCOD of domestic wastewater at 10°C.** (Results of wastewaters having a SCOD<sub>0</sub> of less than 150 mg/l were omitted while calculating the average SCOD removal efficiencies (Table 6.2) to minimise the scatter in the results).

From Figure 6.1, it can be seen that the SCOD removal is highly dependent on the SS<sub>0</sub> and the SCOD<sub>0</sub> of the wastewater. The SCOD removal over 8 hours for Broomhaugh and Sacriston wastewaters was found to be 76 mg/l (SD=22) and 87 mg/l (SD=1) respectively. The corresponding SCOD removal for Durham wastewaters averaged 196 mg/l (SD=9).

**Table 6.2-Average SCOD removal efficiencies at 10°C**

Retention time (hours)	SCOD removal (%)		
	Broomhaugh	Sacriston	Durham
1	15±1*	10±3	20±7
2	31±17	13±9	24±5
3	31±17	16±9	32±7
4	34±17	21±6	35±10
6	36±20	21±4	45±6
8	43±11	26±0	53±4

Note: \*average±SD.

**Table 6.3-Average wastewater characteristics corresponding to the data shown in Table 6.2**

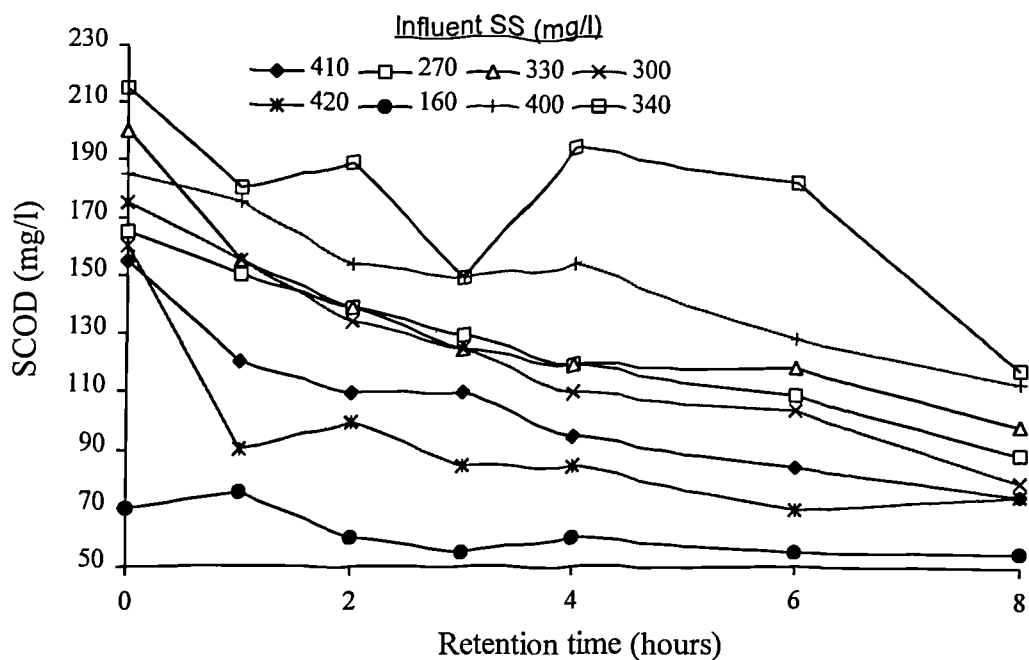
WWTP	SCOD <sub>0</sub> (mg/l)	SS <sub>0</sub> (mg/l)	n
Broomhaugh	175±7	390±42	2
Sacriston	338±4	330±85	2
Durham	372±20	607±6	3

Note : n = number of individual samples studied

For Broomhaugh wastewaters, the influent soluble BOD<sub>5</sub> (SBOD<sub>5</sub>) and SBOD<sub>5</sub> removal over 8 hours were found to be 60 mg/l (SD=5) and 50 % (SD=15) respectively. The corresponding values for Sacriston wastewaters amounted to 138 mg/l (SD=3) and 41% (SD=5). The influent SBOD<sub>5</sub> for Durham averaged 152 mg/l (SD=7) while the removal efficiency over 8 hours averaged 64% (SD=11).

### 6.3.2. Removal of soluble organic matter at 20°C

Figures 6.2 to 6.5 illustrate the variation of SCOD and SBOD<sub>5</sub> with time, for individual wastewaters collected from different WWTPs, during the simulated aerobic gravity transport at 20°C. The corresponding average effluent SCODs are plotted in Figure 6.6. Table 6.4 summarises the average SCOD removal efficiencies for different wastewaters, over a retention period of 1 to 8 hours. The SCOD<sub>0</sub> and SS<sub>0</sub> values corresponding to the results shown in Figure 6.6 and Table 6.4 are shown in Table 6.5. Table 6.6 shows the average SBOD<sub>5</sub> removal efficiencies with corresponding influent wastewater characteristics summarised in Table 6.7.



**Figure 6.2-Variation in SCOD of Broomhaugh wastewater at 20°C** (Results of samples having SS<sub>0</sub> of 160, 420 and 340 mg/l have been omitted while calculating the average SCOD removal efficiencies shown in Table 6.4, to minimise the scatter).

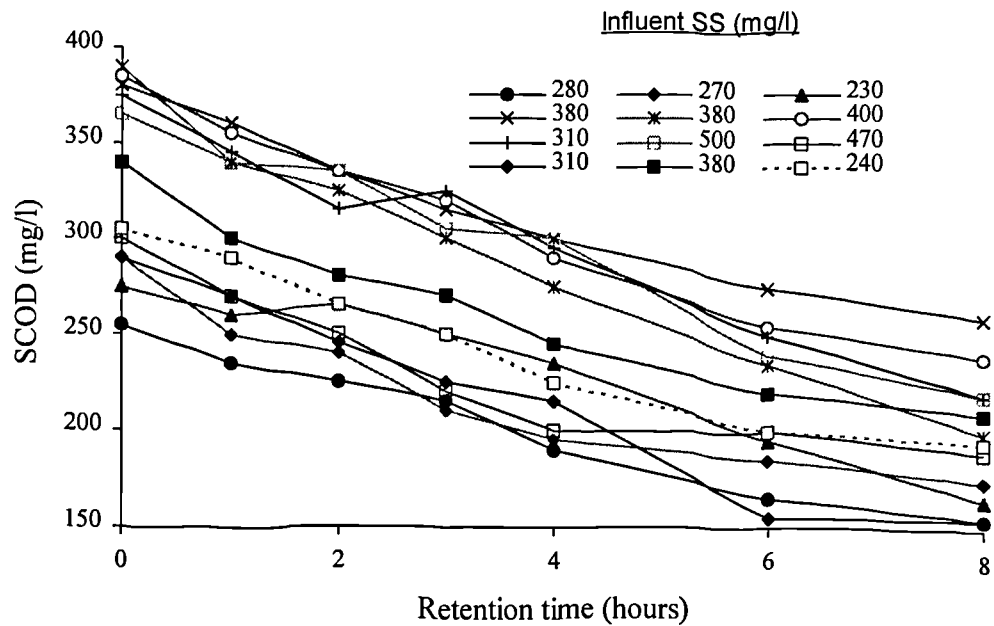


Figure 6.3-Variation in SCOD of Sacriston wastewater at 20°C.

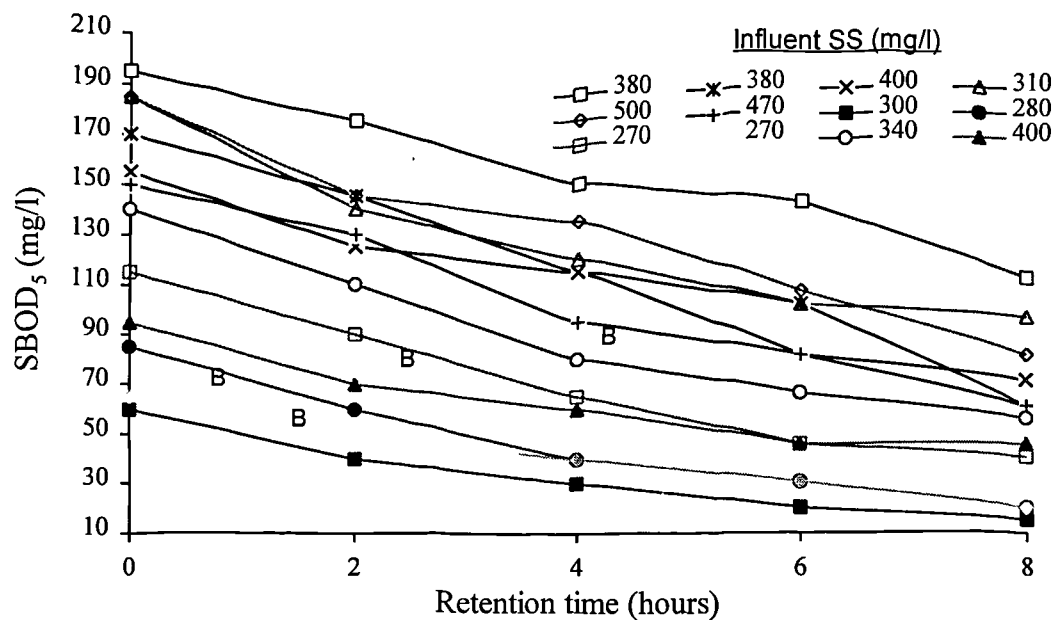


Figure 6.4-Variation in SBOD<sub>5</sub> of Broomhaugh and Sacriston wastewater at 20°C (B represents samples from Broomhaugh WWTP).

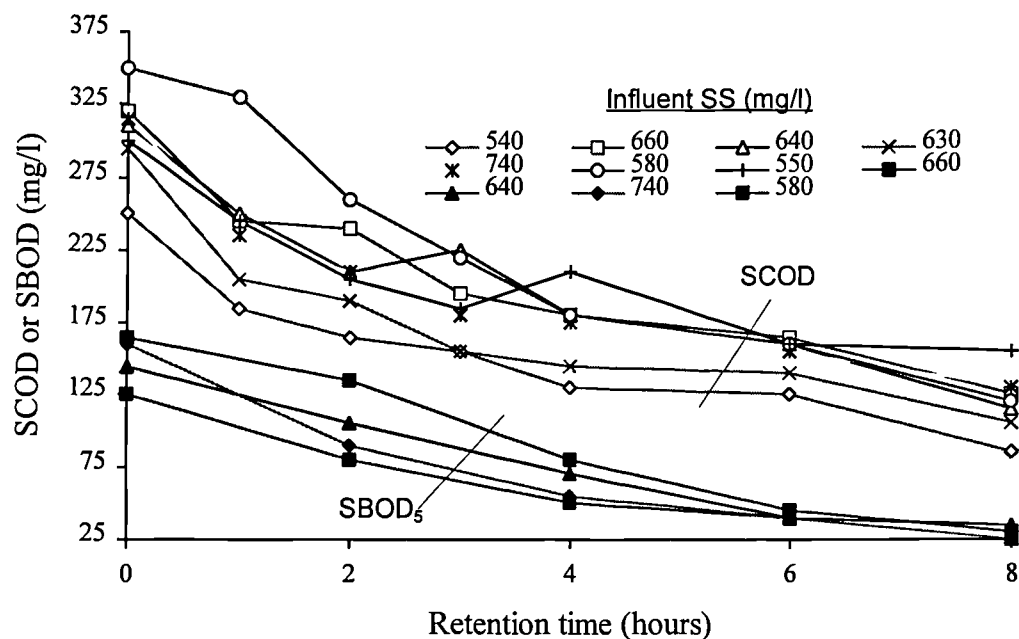


Figure 6.5-Variation in SCOD and SBOD<sub>5</sub> of Durham wastewater at 20°C.

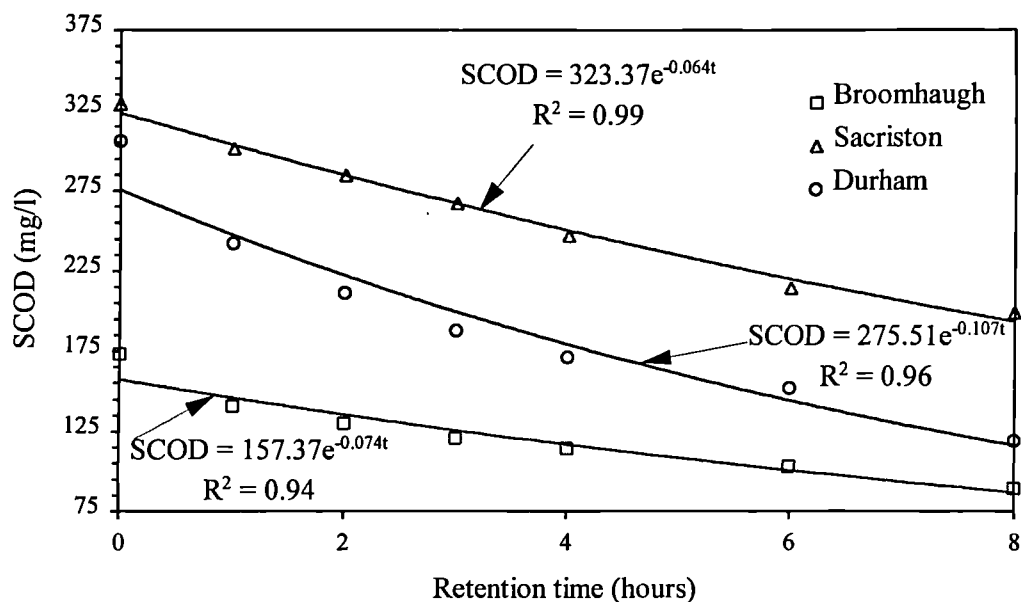


Figure 6.6-Variation in SCOD of domestic at 20°C (Average of n samples (Table 6.5)

are plotted, t = retention time)

**Table 6.4-Average SCOD removal efficiencies at 20°C**

Retention time (hours)	COD removal (%)		
	Broomhaugh	Sacriston	Durham
1	14±8	8±3	21±8
2	23±7	13±4	31±4
3	27±7	19±5	39±6
4	32±10	25±6	44±7
6	38±6	35±5	50±3
8	48±6	40±5	61±6

**Table 6.5-Wastewaters characteristics corresponding to the data shown in Table 6.4**

WWTP	SCOD <sub>0</sub> (mg/l)	SS <sub>0</sub> (mg/l)	n
Broomhaugh	176±17	353±61	5
Sacriston	329±48	346±87	12
Durham	306±30	620±70	7

**Table 6.6-Average soluble BOD<sub>5</sub> removal efficiencies at 20°C**

Retention time (hours)	Average SBOD <sub>5</sub> removal (%)		
	Broohaugh	Sacriston	Durham
2	26±5	19±6	31±11
4	42±5	35±10	57±7
6	57±6	47±11	72±3
8	64±8	59±10	81±4

**Table 6.7-Wastewaters characteristics corresponding to the data shown in Table 6.6**

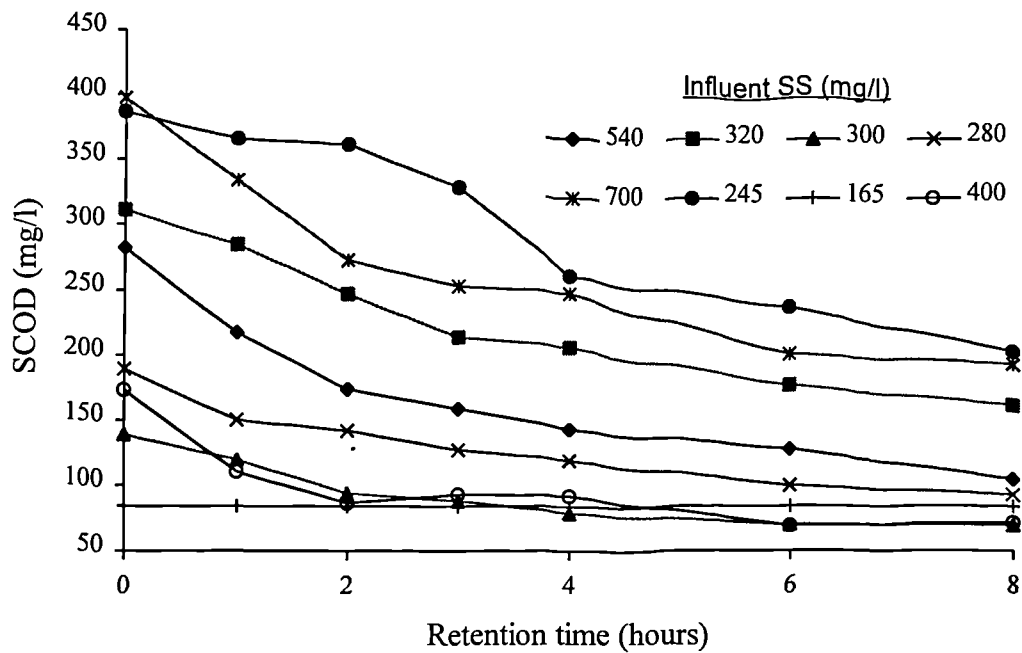
WWTP	SCOD <sub>0</sub> (mg/l)	SBOD <sub>5</sub> (mg/l)	SS <sub>0</sub> (mg/l)	n
Broomhaugh	185±19	90±32	328±49	4
Sacrison	346±47	155±38	374±84	8
Durham	324±18	149±18	655±66	4

The results shown in Figures 6.2-6.6 demonstrate that that the SCOD removal pattern in the wastewater at 20°C would strongly depend upon the initial concentration of soluble organic matter and the suspended solids present in the wastewater.

The SCOD removal over a retention period of 8 hours averaged 84 mg/l (SD=13) for Broomhaugh wastewater. The corresponding SCOD removal for Sacrison and Durahm wastewaters averaged 130 (SD=25), and 186 (SD=27) mg/l, respectively. The average SBOD<sub>5</sub> removal over a retention period of 8 hours averaged 56 (SD=17), 88 (SD=15) and 120 (SD=18) mg/l, for Broomhaugh, Sacrison and Durham wastewaters, respectively.

### 6.3.3. Removal of soluble organic matter at 30°C

Figure 6.7 shows the changes in the SCOD of the individuals wastewater over a retention period of 8 hours. Data summarising the average SCOD removal efficiencies for different wastewaters are listed in Table 6.8, while the corresponding influent wastewater characteristics are shown in Table 6.9.



**Figure 6.7-Variation in SCOD of domestic wastewater at 30°C** (Results for wastewaters having  $SCOD_0$  of less than 150 mg/l were omitted while calculating the average SCOD removal efficiencies).

**Table 6.8-Average SCOD removal efficiencies at 30°C**

Retention time (hours)	SCOD removal efficiency (%)		
	Broomhaugh	Sacriston	Durham
1	23±11	7±2	19±5
2	36±13	14±10	35±5
3	39±7	23±11	40±5
4	42±5	33±1	43±8
6	52±7	41±3	52±4
8	53±5	48±0	57±8



**Table 6.9-Wastewaters characteristics corresponding to the data shown in Table 6.8**

WWTP	SCOD (mg/l)	SS (mg/l)	n
Broomhaugh	167±26	327±64	2
Sacriston	349±54	283±53	2
Durham	340±82	620±113	3

The SCOD removal over a retention period of 8 hours averaged 89 mg/l (SD=17), 166 mg/l (SD=24) and 191 (SD=19) mg/l respectively for Broomhaugh, Sacriston and Durham wastewaters. For Broomhaugh wastewater, the influent SBOD<sub>5</sub> and SBOD<sub>5</sub> removal over 8 hours, were found to be 51 mg/l (SD=12, n=2) and 75% (SD=5), respectively. The corresponding values for Sacriston wastewaters amounted to 142 mg/l (SD=30, n=2) and 73% (SD=3). The influent SBOD<sub>5</sub> for Durham averaged 123 mg/l (SD=15, n=2) while the removal efficiency over 8 hours averaged 87% (SD=4).

#### **6.3.4. Effect of wastewater strength on SCOD removal**

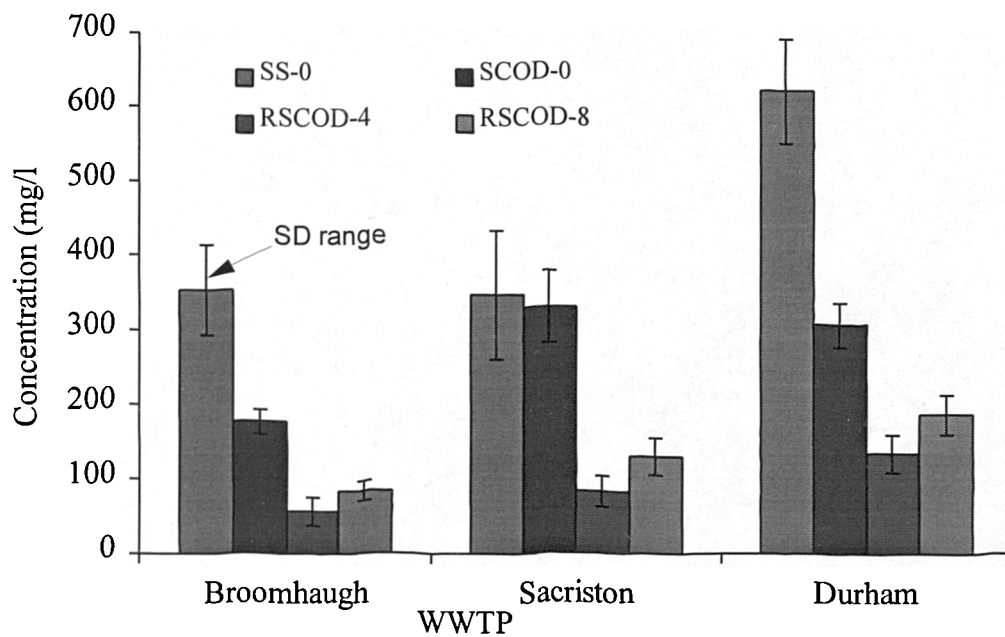
As already indicated in Sections 6.3.1-6.3.3, the SCOD removal from the wastewater at all temperatures investigated in this study, was found to vary as a function of the initial SS (SS<sub>0</sub>) and initial SCOD (SCOD<sub>0</sub>) of the wastewater.

At 10°C, although the data collected did not cover a wide range of wastewater SCODs and SSs (Figure 6.1) in order to statistically assess the effects of SCOD<sub>0</sub> and SS<sub>0</sub> on SCOD removal, the SCOD removal, in general, appeared to increase with an increase in SS<sub>0</sub> and SCOD<sub>0</sub>.

At 10°C, the SCOD removal over 4 and 8 hours for Broomhaugh wastewaters was found to be 38 and 60 mg/l, respectively, for a sample having  $\text{SCOD}_0$  of 170 and  $\text{SS}_0$  of 360 mg/l. The corresponding SCOD removals increased to 83 and 91 mg/l, when the  $\text{SCOD}_0$  and  $\text{SS}_0$  of the wastewater were 180 and 420 mg/l respectively. A very low SCOD removal was observed over 8 hours when the  $\text{SCOD}_0$  was in the range of 85-95 mg/l and the  $\text{SS}_0$  was 150 mg/l. Similarly the SCOD removal appeared to level off and no further removal was observed when the SCOD had reached a value of 90 mg/l, for a wastewater having an  $\text{SCOD}_0$  of 180 mg/l (Figure 6.1).

A comparison of the average SCOD removal efficiencies for Sacriston and Durham wastewaters at 10°C indicates that, on average, a 34 mg/l (10%) increase in  $\text{SCOD}_0$  and a 277 mg/l (84%) increase in  $\text{SS}_0$  increased the amount of SCOD removal for Durham wastewater by 86 and 125%, over a retention period of 4 and 8 hours respectively. From these results, it appears that the  $\text{SS}_0$  (approximating to initial biomass concentration) plays a major role in the removal of SCOD at 10°C.

Figure 6.8 illustrates the changes in average SCOD removals versus average  $\text{SS}_0$  and  $\text{SCOD}_0$  values for different wastewaters at 20°C. The results shown in Figure 6.8 indicate that, in general, the SCOD removal in wastewater increases with an increase in  $\text{SS}_0$  or  $\text{SCOD}_0$  of the wastewater at 20°C. For the Broomhaugh wastewater, the SCOD removal over 8 hours varied from 75-100 mg/l, when the  $\text{SCOD}_0$  and  $\text{SS}_0$  were in the range of 155-200 and 270-410 mg/l, respectively. The corresponding SCOD removal however, decreased to 15 mg/l for a dilute wastewater having an  $\text{SCOD}_0$  and  $\text{SS}_0$  of 70 and 160 mg/l, respectively (Figure 6.2).



**Figure 6.8-The effect of  $SS_0$  and  $SCOD_0$  on SCOD removal at 20°C.** Average of  $n$  samples (Table 6.4) are plotted along with standard deviation (SD).

In order to delineate the effects of  $SCOD_0$  and  $SS_0$  on SCOD removal at 20°C, the SCOD removals for 27 wastewaters over a retention period of 1 to 8 hours were analysed using multiple regression analysis. Appendix C-1 shows how the data was arranged to be used for regression analysis. Computer outputs exhibiting the regression equations for SCOD removals and their summary statistics including the estimated coefficients for the independent variables involved, their standard deviations and t-ratios, are shown in Appendix C-2. An inspection of the summary statistics of the regression analyses revealed that at a significance level of 5%, the SCOD removal was significantly affected ( $p < 0.05$ ) by  $SS_0$  over a retention period of 1 to 3 hours. For higher retention periods ( $\geq 4$  hours), the SCOD removal was significantly affected by both the  $SS_0$  and the  $SCOD_0$  of the wastewater.

Based on all the SCOD data collected at 20°C, the following empirical relationship for determining the amount of SCOD removal at a given retention period under aerobic gravity transport, was formulated

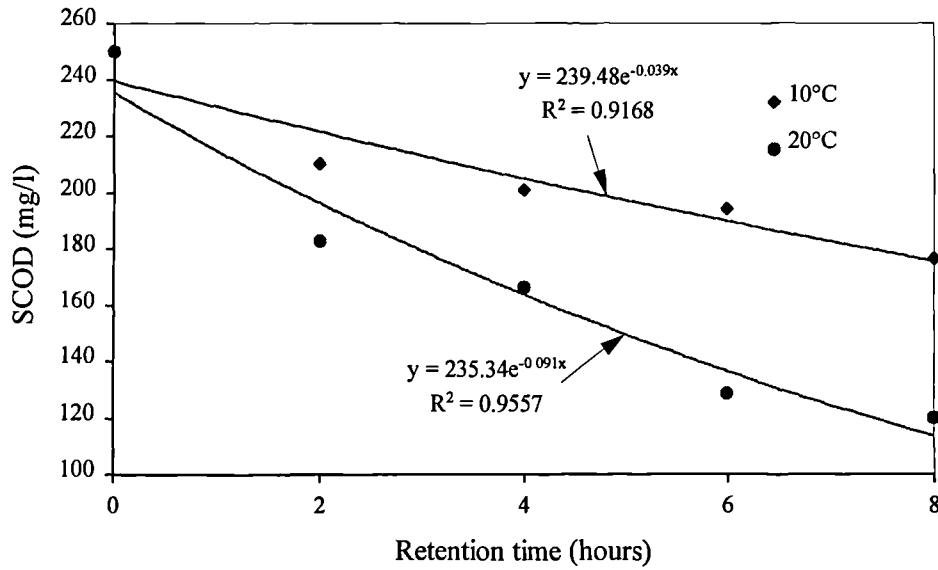
$$\text{RSCOD}_t = -74.4 + 0.156 \text{SS}_0 + 0.146 \text{SCOD}_0 + 12.9 t \quad (R^2 = 0.8) \quad (6.1)$$

Where:  $\text{RSCOD}_t$  = SCOD removal (mg/l) over a retention period of  $t$  (1-8) hours, and  $\text{SS}_0$  and  $\text{SCOD}_0$  are, respectively, influent SS and SCOD concentrations (mg/l), and  $R^2$  = correlation coefficient.

A statistical analysis of the results of 7 wastewaters investigated at 30°C showed that the SCOD removal over the first three hours would follow a trend similar to that at 20°C. The SCOD removal after a retention period of 4 hours however, was in general significantly affected only by  $\text{SCOD}_0$  concentration.

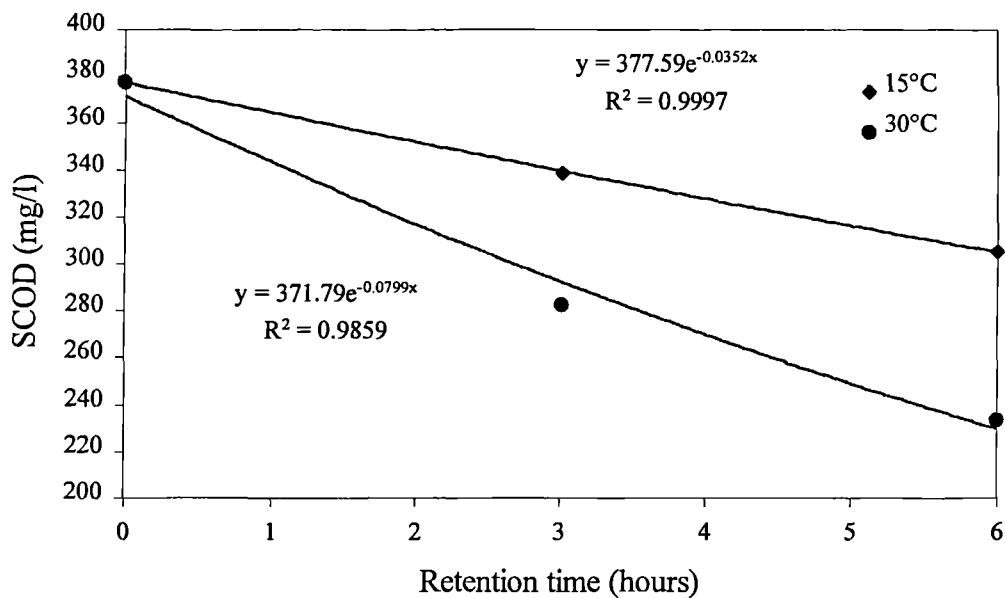
#### **6.3.5. Temperature dependency of SCOD removal from the wastewater**

Higher removal rates are associated with higher temperatures due to the increased metabolic activity at high temperatures. Generally the bacterial reaction rate approximately doubles for each 10°C rise in temperature (Newcombe *et al.*, 1977). To estimate the temperature sensitivity of the SCOD removal rates in the wastewater, experiments were run in parallel on the same wastewater at (i) 10° and 20°C (Figure 6.9), and (ii) 15° and 30°C (Figure 6.10). A total of 8 wastewaters (4 at 10°C and 20°C, and 4 at 15° and 30°C) were subjected to batch tests.



**Figure 6.9-Comparison of SCOD removal in the wastewater at 10° and 20°C**

( $n = 4$ ,  $SS_0 = 540 \pm 250$  mg/l).



**Figure 6.10-Comparison of SCOD removal in the wastewater at 15° and 30°C**

( $n=4$ ,  $SS_0 = 350 \pm 83$  mg/l).

The average first order SCOD removal rate coefficients were used to calculate the temperature rate coefficients from the van't Hoff-Arrhenius relationship which is expressed by the following equation (Benfield and Randall, 1980) :

$$K_2 = K_1 \theta^{(T_2 - T_1)} \quad (6.2)$$

Where  $K_1$  and  $K_2$  are the first order reaction rate coefficients at temperatures  $T_1$  and  $T_2$ , respectively while  $\theta$  is the temperature coefficient.

From the first order SCOD removal rates,  $K_1 = 0.039 \text{ h}^{-1}$  at  $10^\circ\text{C}$ , and  $K_1 = 0.091 \text{ h}^{-1}$  at  $20^\circ\text{C}$  (Figure 6.9),  $\theta$  was found to be 1.088. The  $K_1$  at  $15^\circ$  and  $30^\circ\text{C}$  (Figure 6.10) amounted to  $0.0352$  and  $0.0799 \text{ h}^{-1}$ , respectively. The corresponding value of  $\theta$  was found to be 1.056.

#### 6.4. TOTAL COD REMOVAL

To estimate the changes in total COD (TCOD) in the wastewater during simulated aerobic gravity transport, the TCOD of unsettled wastewater samples was determined before and after 8 hours of oxygen addition. The average TCOD removal efficiencies over a retention period of 8 hours are summarised in Table 6.10.

The average TCOD removal over 8 hours amounted to  $50 \text{ mg/l}$  ( $\text{SD}=12$ ,  $n=4$ ) at  $10^\circ\text{C}$ , when the influent SS was more than  $200 \text{ mg/l}$ . For samples having a SS concentration of less than  $200 \text{ mg/l}$ , one wastewater did not show any change in TCOD over 8 hours while another exhibited an 11% decrease over 8 hours.

**Table 6.10-Average TCOD removal efficiencies\***

Temperature	Influent SS	Influent TCOD	TCOD removal	n
°C	(mg/l)	(mg/l)	(%)	
10	468±166	1140±335	5±3	4
20	431±153	871±250	15±6	14
30	428±185	890±349	15±4	5

Note: \* : average results for samples showing a decrease in TCOD over 8 hours

The TCOD removal over 8 hours at 30°C averaged 136 mg/l (SD=74, n=5). The corresponding TCOD removal at 20°C amounted to 110 mg/l (SD=51, n=14). At 20°C, one wastewater did not demonstrate any change in TCOD over 8 hours, while the effluent TCOD of two samples was slightly higher than the corresponding influent. At all the temperatures investigated, the TCOD removal over 8 hours did not appear to show any relation to the influent TCOD and SS concentrations for the range investigated in this study.

## 6.5. EFFECT OF OXYGEN ADDITION ON SS IN THE WASTEWATER

To study the changes in SS concentration (biomass concentration) in the wastewater during simulated aerobic gravity transport, the SSs of the wastewater were measured before and after six hours of oxygen addition in the batch reactor. The average changes in SS of the wastewater, as a result of oxygen addition for 6 hours, are given in Table 6.11.

**Table 6.11-Average changes in SS of wastewater**

Temperature °C	Influent SCOD (mg/l)	Influent SS (mg/l)	Change in SS (%)	n
10	243±126	374±187	5±7	10
20	271±86	417±153	5±11	25
30	245±116	369±174	10±12	8

A portion of the effluent from the batch reactor, after a retention period of 6 hours was settled in a one-litre cylinder for one hour and the SS of the supernatant was determined. The corresponding SS removal efficiencies at different temperatures are summarised in Table 6.12. The results shown in Table 6.12 indicate that the SS removal efficiencies in the bench-scale settling column are significantly higher than those achieved in a typical primary sedimentation tank (50-70%), thus reflecting the improved primary settleability due to aerobic conditions during the simulated gravity transport.

**Table 6.12-SS removal efficiencies**

Temperature °C	Effluent SS (mg/l)	Supernatant SS (mg/l)	SS removal (%)	n
10	444±224	96±49	77±6	7
20	406±216	73±39	80±10	7
30	343±192	79±53	78±4	6

From Table 6.11, it can be seen that the changes in SS (biomass growth) as a result of oxygen addition showed a wide variation. All but one wastewater collected from Durham WWTP showed an increase in SS at all the temperatures investigated. The wastewaters collected from other WWTPs however, did not always display a similar trend. Out of 10

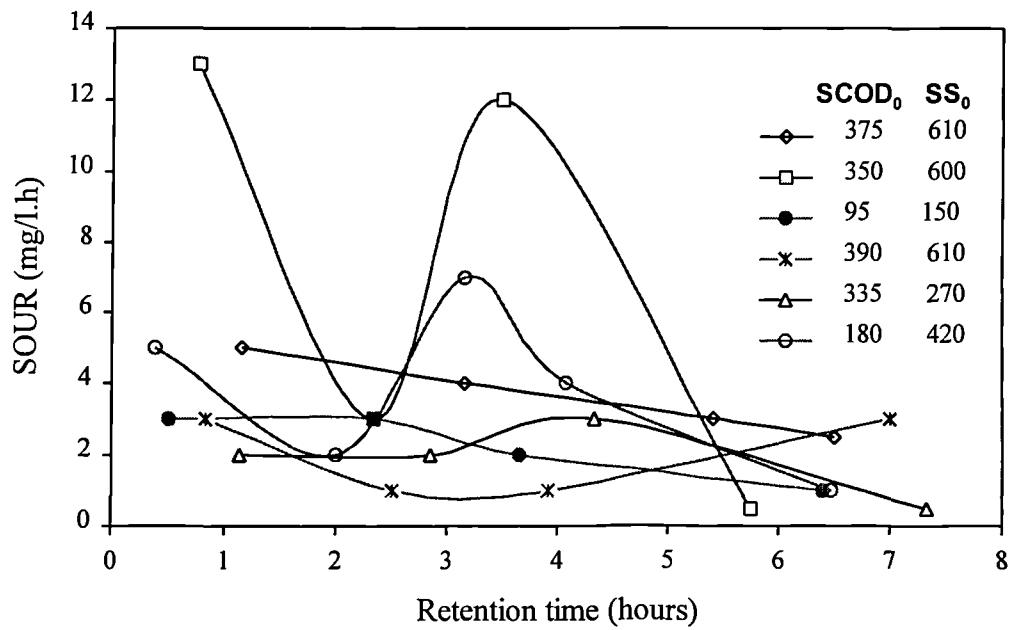


wastewaters investigated for changes in SS at 10°C, 30% either did not exhibit any change in SS or showed a decrease in SS as a result of oxygen addition for 6 hours. The corresponding figures at 20 and 30°C were 29% (7 out of 27) and 25% (2 out of 8). The overall decrease in SS ranged from 3-28%. The change in SS of the wastewater did not appear to show any obvious relation to the changes in TCOD. It appeared however that, in general, dilute wastewaters having low SCODs and low SS (especially those collected from Broomhaugh) were more likely to exhibit a decrease in SS as a result of oxygen addition. The VSS of the wastewater also followed a trend similar to the SS with regard to changes due to oxygen addition.

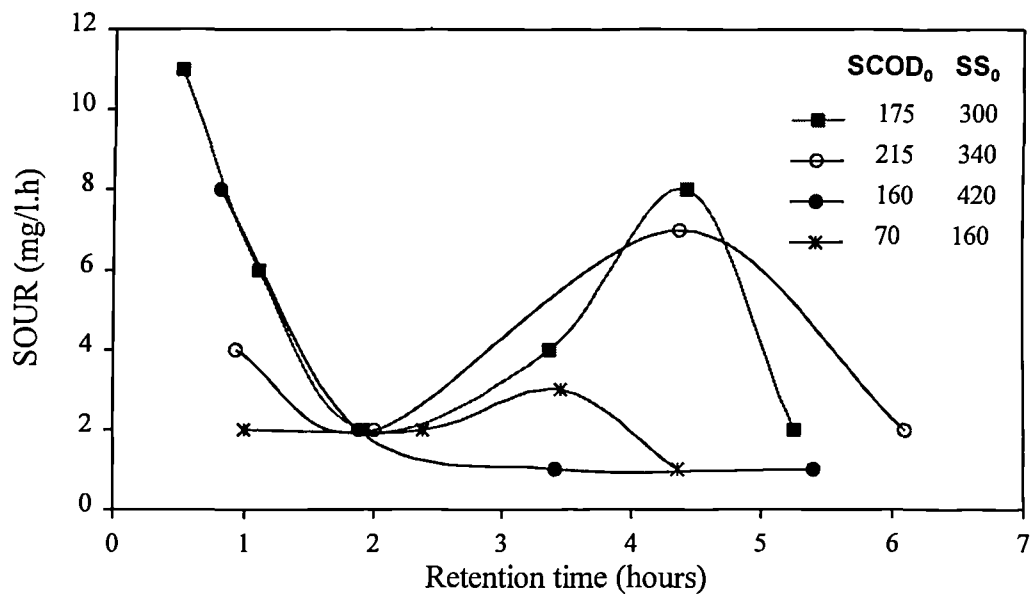
For the wastewaters which showed an increase in SS (or VSS) over 6 hours, the yield coefficient, calculated from changes in VSS of the wastewaters and the corresponding SCOD removals (Appendex D-1) varied from 0.1-0.35 at 10°C, 0.1-0.72 at 20°C and 0.1-0.5 mg VSS/mg SCOD at 30°C. The value of the yield coefficient averaged 0.3 mg VSS/mg SCOD (SD=0.17, n=15) at 20°C. This value is lower than the literature values of the yield coefficient of 0.4-0.8 mg VSS/mg SCOD removed for the activated sludge process (Metcalf and Eddy, 1991).

## **6.6. OXYGEN UPTAKE RATE**

To estimate the oxygen uptake rate of the suspended biomass present in the wastewater (SOUR), samples of the oxygenated wastewater were taken from the batch reactor at specified intervals of time and were subjected to SOUR tests (Section 4.3.1). Figures 6.11-6.15 show the variation in SOUR of the wastewaters at the different temperatures investigated.



**Figure 6.11- Variation in SOUR of wastewater at 10°C (SCOD<sub>0</sub> and SS<sub>0</sub> are expressed in mg/l).**



**Figure 6.12- Variation in SOUR of the Broomhaugh wastewater at 20°C.**

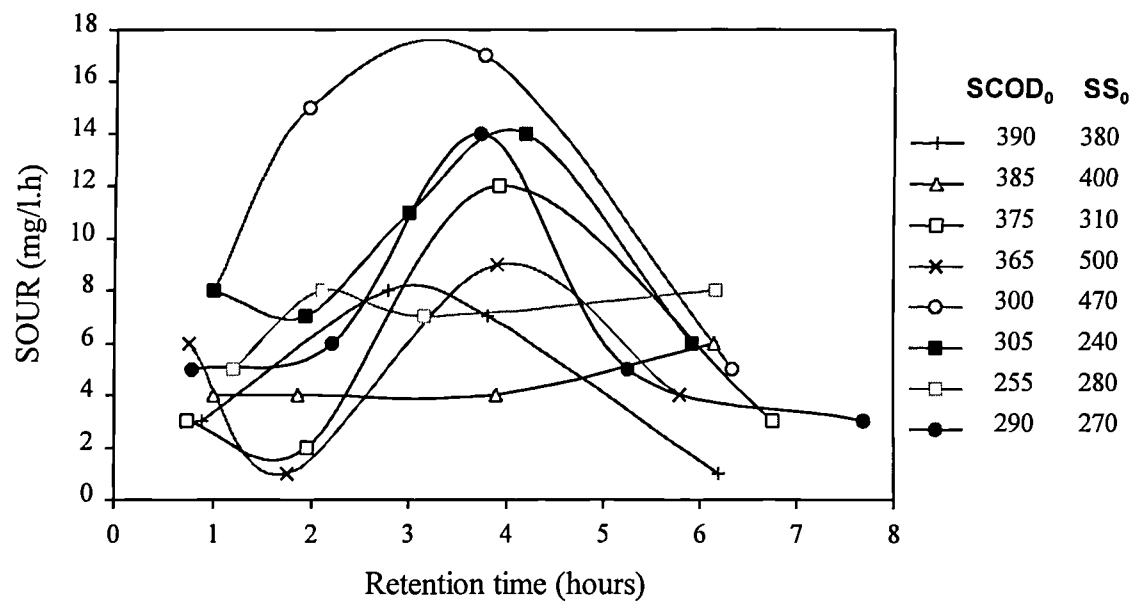


Figure 6.13-Variation in SOUR of the Sacriston wastewater at 20°C.

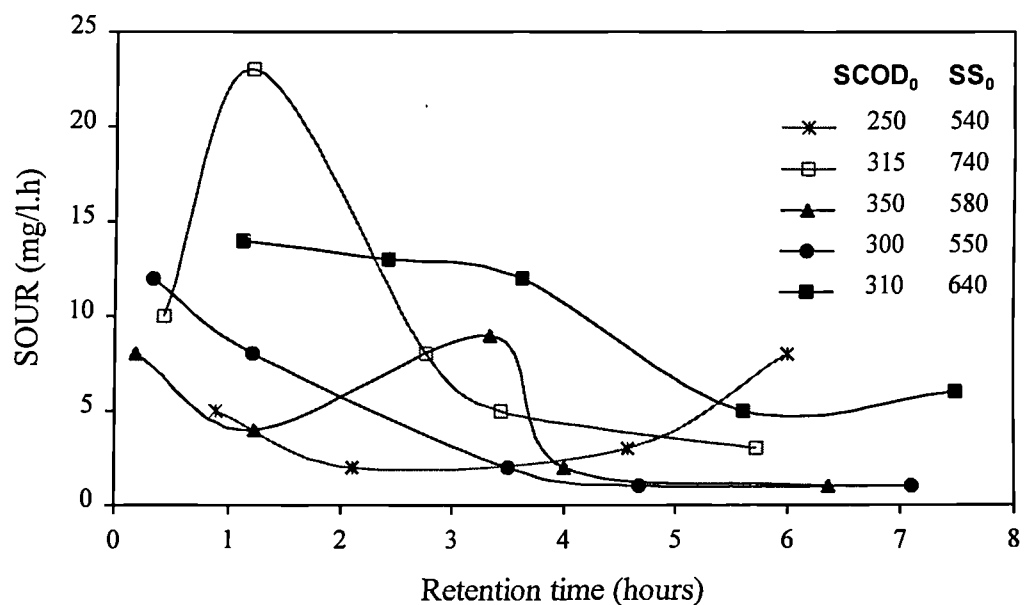
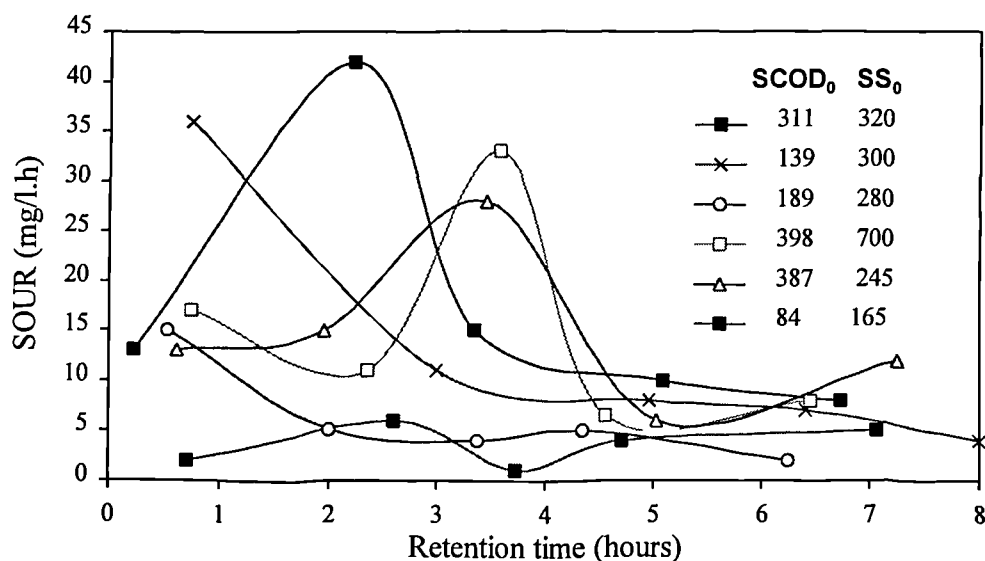


Figure 6.14-Variation in SOUR of the Durham wastewater at 20°C.



**Figure 6.15-Variation in SOUR of domestic wastewater at 30°C.**

From the data presented in Figures 6.11-6.15, the following three distinct types of time profiles of SOUR could be identified:

- (i) Type 1: SOUR is low initially, increases to a peak value and then declines,
- (ii) Type 2: SOUR is at a maximum initially, then declines with the passage of time,
- and
- (iii) Type 3: the SOUR does not show any general trend with respect to time.

When compared to the samples collected from Broomhaugh and Durham WWTPs which displayed both Type-2 and Type-3 profiles, most of the samples taken from Sacriston appeared to show a regular trend (Type-1 profile) with respect to time. The initial (after about 1 hour from the start of the test) and peak values of SOUR at 20°C ranged from 2-8 and 8-17 mg/l.h, respectively. The corresponding value of SOUR after a retention period of about 6 hours varied from 1-6 mg/l.h. The initial SOUR value at 30°C was found to be about 13 mg/l, while peak and final values varied from 28-42, and 8-12 mg/l.h,

respectively. The retention time at which the peak value of SOUR occurred varied from 2-5 hours.

As can be seen from Figures 6.11-6.15, the SOUR values for wastewaters collected from individual WWTPs varied widely, and did not appear to show any clear relation with the corresponding  $SS_0$  or  $SCOD_0$  concentrations. The SOUR values however, were quite low when the  $SCOD_0$  of the wastewater was  $\leq 110$ , 70, and  $\leq 189$  mg/l at 10, 20 and 30°C, respectively. This indicates that, at a given temperature, dilute wastewaters (having  $SCOD_0$  concentrations similar to the above) are likely to exhibit lower SOUR values. Maximum SOUR values of 13, 25 and 40 mg/l.h were observed at 10, 20 and 30°C, respectively, indicating that the SOUR of the wastewater would be significantly affected by temperature.

## 6.7. DISCUSSION

### 6.7.1. SCOD removal

The results of the present study clearly demonstrate that, by maintaining aerobic conditions in sewers, a significant degree of soluble organic matter could be achieved. These results support the conclusions of previous field investigations (Pomeroy and Parkhurst, 1972; Newcombe *et al.*, 1979) that sewers could be used to effect at least partial treatment of wastewater during transit.

The results presented herein indicate that the removal of soluble organic matter in wastewater in sewers would be strongly affected by the strength of the wastewater. At a given temperature, a wastewater with a high initial SS ( $SS_0$ ) and SCOD ( $SCOD_0$ ) was

found to achieve a high degree of SCOD removal while compared to a wastewater having low  $\text{SCOD}_0$  and  $\text{SS}_0$ . The results also demonstrate that, if the sewers are to be used as an aerobic treatment facility, the effluent quality of such a system may fluctuate greatly, depending upon the diurnal and seasonal variations in the influent concentrations.

During the first 3 to 4 hours from the start of the batch experiments, the SCOD removal rates at 20°C and 30°C were found to be greatly affected by the  $\text{SS}_0$  concentration and to be independent of the  $\text{SCOD}_0$  concentration. A possible explanation for this phenomenon may be that during the first few hours of oxygen addition, the surface of the bacteria is completely saturated with substrate and all enzymes are in a complexed state (Sundstrom and Klei, 1979). An increase in SCOD removal with an increase in initial SS concentration of the wastewater was expected due to the high number of micro-organisms present in the wastewater which were able to metabolise the organic matter. The dependency of the SCOD concentration on SCOD removal during the latter stages of the experiments confirms the suggestion of Benfield and Randall (1980) that at low substrate concentrations in batch cultures, the rate of substrate utilisation would be directly proportional to the substrate concentration, because very little substrate is available to complex with the enzymes.

The removal of soluble organic matter from the wastewater flowing in the sewers, while taking into account the effect of changes in wastewater strength, has not been reported previously. Nevertheless, the findings of the present study are in reasonable agreement with those few reported in the literature. Lokkeggard *et al.* (1995) reported a soluble COD removal of 35 mg/l over 8 hours, for a wastewater having a  $\text{SCOD}_0$  and  $\text{SS}_0$  of 140 and 160 mg/l, respectively, at 17°C. The present study noted a similar trend for dilute

wastewaters. Henze (1992) based on laboratory studies, reported a soluble COD removal of about 50 mg/l over a retention period of 3 hours, for a freshly produced wastewater having a  $SCOD_0$  of about 175 mg/l. This removal rate is of the same order of the magnitude as that found in present study for a wastewater having a  $SCOD_0$  of 175 mg/l and a  $SS_0$  of 300 mg/l (Figure 6.2).

The SCOD results (Section 6.3) showed that the SCOD removal in the wastewater was at a maximum during the initial hours of oxygen addition. Of all the wastewaters investigated at 20°C, 89 percent showed a maximum SCOD removal during the first hour of oxygen addition, while the corresponding time for the remaining 11 percent was 2 hours. The SCOD removal rate during the first 2 hours averaged 19, 25 and 47 mg/l.h for Broomhaugh, Sacriston and Durham wastewaters, respectively. The corresponding rate during the next 6 hours averaged 10, 15 and 15 mg/l.h, respectively. These results are in good agreement with those reported by Lokkegaard *et al.* (1995). They, based on batch studies undertaken to investigate the kinetics of the wastewater, found that the readily biodegradable organic matter in most of the wastewaters studied was depleted during the first two hours under aerobic conditions.

A rapid decrease in SCOD during the first few hours could be due to the high degradation rate associated with readily biodegradable organic matter (Henze, 1992) while the subsequent decrease in SCOD removal rate may be attributed to the hydrolysis of slowly biodegradable organic matter (Benfield and Randall, 1980; Raunkjaer *et al.* 1993). Henze (1992) reported a biodegradable SCOD fraction equal to 35% of the total SCOD for a wastewater with a  $SCOD_0$  of 170 mg/l. For a similar wastewater, Lokkegaard *et al.* (1995) reported the readily biodegradable fraction to be 18% of the initial SCOD. Lesouef *et al.*

(1992) have reported the readily biodegradable fraction to be 25 and 30% for two wastewaters having total CODs of 400 mg/l. In the present study, an average SCOD removal of 23 % (SD=7) was observed at 20°C for Broomhaugh wastewater (Table 6.4) having an average SCOD of 176 mg/l (SD=17). This removal efficiency is reasonably consistent with the ranges of the readily biodegradable fractions of SCOD reported in the above studies.

When the change in SCOD over 8 hours for individual wastewaters investigated at 10°-30°C was described as zero- and first-order reactions (Appendix C-3), the first-order fits yielded higher correlation coefficients than the corresponding zero-order values. This suggests that the changes in SCOD in the wastewater during simulated aerobic gravity transport would be best described by first-order kinetics (Figure 6.6). This observation is in agreement with other published results for the activated sludge processes (Benfield and Randall, 1980; Argaman, 1991).

The results of the current study suggest that the retention time required to achieve a certain degree of soluble organic matter removal in the wastewater under aerobic conditions would depend upon the strength of the wastewater ( $SCOD_0$  and  $SS_0$ ) and the temperature. This is in contrast to the observation by Shaw (1981) that a retention period of about 20 hours would be required to achieve a high degree of self-purification in sewers.

Ozer and Kasigra (1995) pointed out that, when compared with low substrate concentrations, higher substrate concentrations in wastewater would result in higher reaction and respiration rates, which would eventually result in the adequacy of shorter lengths of sewers to achieve an equal decrease in concentration. The present study also



demonstrated this effect. For Durham wastewater (having a high  $SCOD_0$  and  $SS_0$ ), the SCOD removal over a retention period of 1 hour averaged 64 mg/l, at 20°C. The corresponding retention time required to achieve a similar amount of SCOD removal for Broomhaugh wastewater (low  $SS_0$  and low  $SCOD_0$ ) was found to be 6 hours.

From Figures 6.1-6.5 it can be seen that a few wastewaters, especially those collected from Broomhaugh (Figure 6.2) showed a slight increase in SCOD over the course of the time, thus offsetting to some extent the early decrease in SCOD, and producing a saw-toothed profile of SCOD variation over time. A similar effect has been reported by Raunkjaer *et al.* (1995). They, while measuring the dissolved COD of raw wastewater at four stations along a gravity sewer, found that a few samples showed an increase in dissolved COD between two intermediate sampling stations.

A possible reason for the increase in SCOD mentioned above could be the contribution of the SCOD as a result of hydrolysis of the slowly biodegradable fraction. When compared with the readily biodegradable fraction of SCOD, which can pass directly through the cell wall for synthesis and oxidative metabolism by organism, the slowly biodegradable substrate is removed from the wastewater by adsorption onto the floc surface and by enmeshment in the floc structure. This slowly biodegradable material is then hydrolysed thus causing an increase in soluble COD (Benfield and Randall, 1980).

For Durham wastewater (which had a high  $SS_0$  and high  $SCOD_0$ ) the  $SBOD_5$  removal efficiency over a retention period of 8 hours averaged 64%, 81% and 87% at 10°, 20° and 30°C, respectively. When compared with Broomhaugh and Sacriston wastewaters, a high

removal efficiency for Durham wastewater at all temperatures studied suggests that by increasing the  $SS_0$  concentration of the wastewater in sewers (by seeding it with activated sludge), a removal efficiency comparable to that of secondary treatment may be achieved.

The temperature dependency of the SCOD removal rate in the raw wastewater does not seem to have been reported previously. The values of temperature coefficients obtained in the present study (1.088 for 10°-20°C and 1.056 for 15°-30°C) however, compare favourably to those for normal aerobic biological processes reported in the literature. According to Tucek *et al.* (1971), values of temperature coefficient varying from 1.035-1.109 (and even more in the lower temperature region) have been confirmed by other researchers. Benfield and Randall (1980) have reported temperature coefficients of 1.135 and 1.056 for a temperature range of 4-20°C and 20-30°C, respectively.

#### **6.7.2. Total COD removal**

The results shown in Table 6.10 indicate that, when compared with SCOD removal, the TCOD removal in the wastewater during aerobic transport would be much lower. Previous studies have reported a similar effect. Based on field studies, Newcombe *et al.* (1979) reported that during aerobic transport in a pressure main, the changes in total  $BOD_5$  were not significant. Greenfield *et al.* (1980) also reported similar results. Bentzen *et al.* (1995) found that controlled dosing of nitrates to a rising main to control septicity resulted in an increased removal of soluble organic matter, but the corresponding changes in total  $BOD_5$  were not significant. The above mentioned studies argued that the concentration of total organic matter during the aerobic transport would not change significantly due to the

transformation of soluble organic matter into additional biomass which would contribute to total organic matter.

Henze (1992) reported a total COD removal pattern in the wastewater similar to that of SCOD removal. From the results of the present study, there was no evidence to suggest that the TCOD removal in the wastewater would display a removal trend similar to that of the SCOD.

### **6.7.3. SS results**

The SS results (Section 6.5) showed that, in general, the addition of oxygen to the wastewater for 6 hours resulted in an increase in SS (or VSS) concentration. This increase, however fluctuated considerably varying from negligible to 30%, and did not show any clear relation with the initial SCOD to initial VSS ratio. The corresponding yield coefficient varied widely and was, in general, significantly lower than typical values for aerobic processes.

Unexpectedly, 12 out of 45 wastewaters studied within a temperature range of 10°-30°C, either showed a decrease in SS or did not exhibit any change at all. The exact reason for this phenomenon could not be explained. It may be that the shear force produced by the impeller and by the bubbling of oxygen in the reactor had caused some of the solids to dissolve in the wastewater. A decrease in the SS content of the wastewater due to attachment of SS to the walls of the reactor was ruled out as such a phenomenon was not observed.

A low growth or a zero change in biomass concentration observed in batch reactors could also be explained by the ratio of initial substrate ( $SCOD_0$  or  $S_0$ ) to initial biomass concentration ( $X_0$ ) i.e.  $S_0/X_0$  in the batch reactor. The  $S_0/X_0$  or F/M (food/micro-organism or VSS) ratio in the present study varied from 0.46-1.6 mg VSS/mg SCOD. According to Chudoba *et al.* (1992), the  $S_0/X_0$  ratio is the most important factor effecting the growth rate in batch reactors as it determines whether or not cell multiplication will take place. At low  $S_0/X_0$  ratios, the initial energy level is low, and the increase in cell mass reflects the increase in molecular polymer content in the biomass. Consequently, the weight changes may not reflect similar changes in cell number. Based on batch experiments involving a mixture of peptone and glucose, Chudoba *et al.* (1992) noted that at  $S_0/X_0$  ratios of less than 3, the substrates were removed linearly, indicating no cell multiplication. In another study (Chudoba, 1985), it was indicated that in batch cultivation of activated sludge, significant growth would occur only when  $S_0/X_0$  is 10 or more.

The results with respect to changes in SS (Table 6.11) agree with those of many previous studies. Newcombe *et al.* (1979) noted that no significant changes in SS occurred as a result of the addition of oxygen to wastewater in an 8.2 km long pressure main. They found that over a retention period of 2.4-2.8 hours, the SS of the wastewater increased by 1% in a temperature range of 10-12°C and decreased by 2-3% when the temperature varied from 19-20°C. Greenfield *et al.* (1980) observed a two to five-fold increase in the number of viable organisms over 8 hours as a result of the addition of oxygen to raw wastewater in a batch reactor, although the corresponding changes in SS did not show any clear pattern. Barber *et al.* (1980) reported that the addition of oxygen to wastewater in batch reactors resulted in an increase in SS concentration together with higher bacterial

plate counts. They however, noted that a higher number of micro-organisms would account for only a minor fraction of the increase in SS.

The findings of the present study however, are in contrast to the results of Bentzen *et al.* (1995) who reported a 54% increase in SS as a result of the addition of nitrates to a rising main having a retention period of 5.4 hours in a temperature range of 12-22°C. They attributed the increase in SS to the transformation of soluble organic matter into a suspended component. As compared with the results of Bentzen *et al.* (1995), the maximum increase in SS observed in the present study amounted to 11, 19 and 30% at 10, 20 and 30°C, respectively.

#### **6.7.4. Oxygen uptake rate**

A wide variation in SOUR values of the individual wastewaters investigated in the present study may be attributed to the differences in nature and origin of the wastewaters as well as the diurnal changes in transportation time (age of the wastewater), and the prevailing redox values and climatic conditions in sewers.

Previous studies have also reported a great variation in SOUR. Boon and Lister (1975) reported SOUR values of 10-18 mg/l.h for domestic wastewater at a temperature of 14-16°C. The total COD of the wastewater varied from 180-1470 mg/l. Barber *et al.* (1980) reported that the results of SOUR tests were not reproducible and values ranged from 8-18 mg/l with an average of 13 mg/l.h (at 25.5°C). Pescod and Price (1981) reported that the SOUR values of the wastewater entering a treatment plant were found to vary with time of the day, and ranged from 1.5-19.9 mg/l.h.

The SOUR of a domestic wastewater has been found to vary with the age of the wastewater. The SOUR of fresh domestic wastewater under aerobic conditions is low near its origin (2-3 mg/l.h at 15°C), which may increase to a peak value of as high as 20 mg/l.h as the wastewater ages during transportation in sewer, and then declines (Section 2.3.1). Considering the above and the fact that the wastewater samples used in the current study were collected from the inlet to the WWTPs, different SOUR profiles were expected because the age of the individual wastewater samples was likely to change greatly due to changes in the retention time in sewers caused by flow variations.

The majority of wastewaters collected from Sacriston exhibited Type-1 profiles which are identical to the SOUR profiles for young wastewaters reported by Pomeroy and Parkhurst (1972) and Newcombe *et al.* (1979). These results indicate that when compared with wastewaters collected from other WWTPs, the samples taken from Sacriston were relatively young. On the other hand, samples which displayed Type-2 profiles could be relatively aged and might have passed the peak SOUR during transportation to the WWTP. Pomeroy and Parkhurst (1972) also reported SOUR profile, identical to the Type-2 profile observed in the current study, for a wastewater which was collected after being transported in a sewer for about 3.5 hours.

From Figures 6.11 and 6.15, it can be seen that several wastewaters exhibited pronounced secondary SOUR peaks after high initial SOUR values. Similar results were reported by Pomeroy and Parkhurst (1972) and Marwood (1984). Marwood (1984) attributed the initial high SOUR of the oxygenated, previously anaerobic wastewater, to rapid oxidation of sulphide and other minor chemicals and also to biological uptake. He suggested that the

rapid increase in SOUR after the high initial SOUR was due to acclimatisation of facultative aerobic bacteria.

A possible reason for the secondary peaks in SOUR may also be the increase in oxygen demand due to the release of readily biodegradable SCOD to the reactor as a result of the hydrolysis of particulate organic matter (Section 6.7.1). A comparison of the SOUR and SCOD profiles of the wastewaters (Figures 6.1-6.5) indicated that, in general, an increase in SOUR of the wastewater was consistent with an increase in SCOD in the reactor. Wastewaters which produced a zig-zag SOUR profile, also generally exhibited a similar trend with respect to SCOD, thus supporting the hypothesis that the secondary peaks were possibly caused by hydrolysis of the particulate organic matter.

As already mentioned, the SOUR values did not relate clearly with initial SS or SCOD values of the wastewater. This is in agreement with the findings of Pomeroy and Parkhurst (1972), who noted that 1-hour SOUR of wastewater did not bear any relationship to the  $BOD_1$  and  $BOD_5$  of the wastewater. Boon and Lister (1975) and Barber *et al.* (1980) have also reported SOUR results which did not show any clear relation with the total COD or BOD of the wastewater. The SOUR data presented by Boon and Lister (1975), when regressed by this author, against the corresponding total COD values, yielded a correlation coefficient of 12%. Similarly the correlation coefficient between the SOUR and  $BOD_5$  data reported by Barber *et al.* (1980) amounted to 2.7%. The poor correlation of SOUR with COD or  $BOD_5$  of the wastewater implies that there are other factors (Section 2.3.1) which also affect the SOUR of the wastewater.

## CHAPTER SEVEN

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### ADDITION OF OXYGEN TO SEEDED WASTEWATER

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#### 7.1. INTRODUCTION

The results of the experimental work described in Chapters 5 and 6 have clearly demonstrated that by maintaining aerobic conditions in sewers, the indigenous suspended biomass present in the flowing wastewater could be exploited to achieve a significant degree of in-sewer treatment so that wastewater would reach the end of the sewer at least partially treated. In cases, where sufficient retention time is not available however, or where a high degree of in-sewer treatment is required, it has been suggested that active biomass be added to the sewer to enhance the in-line purification (Koch and Zandi, 1973; Pomeroy and Lofy, 1977; Shaw, 1981).

The work described in this chapter was designed to explore the possibility of enhancing in-sewer aerobic treatment by the addition of the activated sludge at the inlet of the sewer. To simulate an aerobic linear gravity sewer with activated sludge being added at the inlet, raw wastewater was added with a specified concentration of activated sludge (Equation 4.1), and maintained aerobic in a batch reactor at 20°C and at atmospheric pressure. The experimental methodology used for this study has been described in detail in Section 4.3.1.

#### 7.2. CHARACTERISTICS OF WASTEWATER AND SEED SLUDGE

The removal of soluble organic matter during simulated aerobic transport was studied at five different initial seed (activated sludge) concentrations i.e. 100, 250, 500, 750 and 1000 mg VSS/l. A minimum of six wastewaters were investigated at each seed concentration. The wastewater used in this study was collected from the inlet to Sacriston WWTP while



the returned activated sludge used for seeding purposes was collected from Durham WWTP. The average characteristics of the wastewaters studied at different seed concentrations are shown in Table 7.1. Due to large number of samples involved, it was not possible to undertake BOD<sub>5</sub> analyses for all the experimental runs. Table 7.2 shows the average characteristics of the wastewaters for which BOD<sub>5</sub> analyses were carried out. The average characteristics of seed sludge are summarised in Table 7.3.

**Table 7.1-Average characteristics of wastewater**

Seed concentration (mg VSS/l)	Wastewater characteristics (mg/l)		Initial SCOD in reactor (mg/l)	n
	SS	SCOD		
100	477±126*	303±69	298±67	9
250	477±126	303±69	289±61	9
500	412±73	324±67	298±60	6
750	382±80	323±67	300±53	6
1000	548±69	339±101	312±79	6

Notes: (i)\* average±SD  
(ii) SCOD=Soluble COD  
(iii) n=number of wastewaters studied

**Table 7.2-Average characteristics of wastewaters for which BOD<sub>5</sub> tests were carried out**

Seed concentration (mg VSS/l)	SS	Initial SBOD <sub>5</sub> in reactor (mg/l)	n
	(mg/l)		
100	486±170*	139±29	5
250	486±170	133±27	5
500	403±95	152±18	3
750	375±96	138±17	4
1000	548±69	137±40	6

Note: SBOD<sub>5</sub> = Soluble BOD<sub>5</sub>

**Table 7.3-Average characteristics of seed sludge**

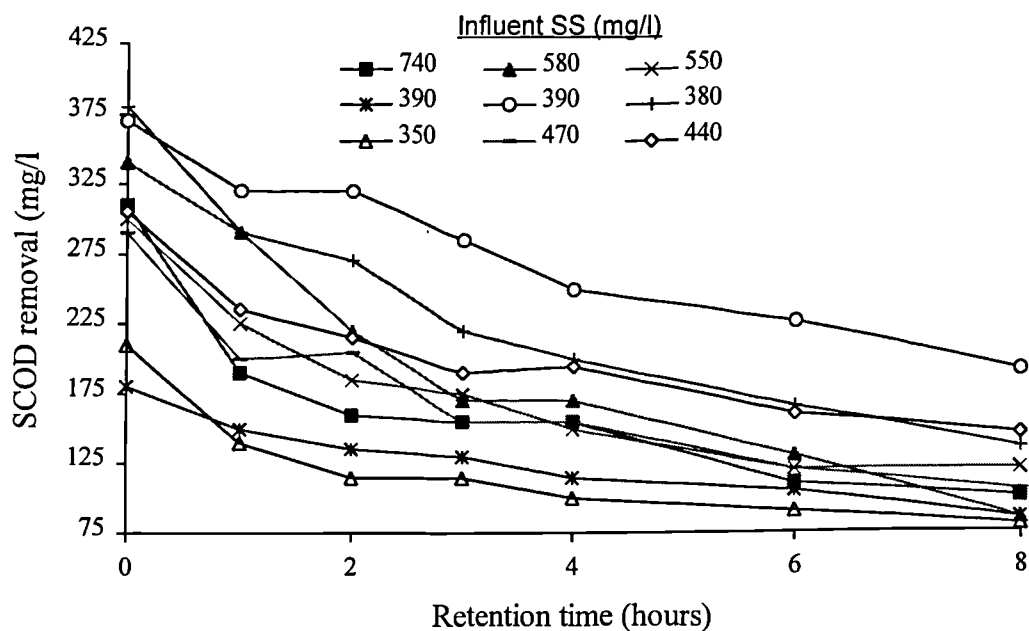
Seed concentration (mg VSS/l)	Seed sludge characteristics (mg/l)		n
	VSS	SCOD	
100	3081±873	162±46	9
250	3081±873	162±46	9
500	3425±1214	183±50	6
750	3928±875	228±100	6
1000	4098±418	210±35	6

As a result of the addition to the wastewater of activated sludge which had a low SCOD, the initial SCOD (or SBOD<sub>5</sub>) in the batch reactor at the start of the test was lower than the corresponding SCOD (or SBOD<sub>5</sub>) of the raw wastewater, depending upon the volume of activated sludge added to the reactor. The initial SCODs and soluble BOD<sub>5</sub> (SBOD<sub>5</sub>) concentrations in the reactor, corresponding to each seed concentration, are also shown in Table 7.1 and 7.2, respectively. The characteristics of the individual wastewaters investigated at each seed concentration are shown in Appendix D-1. Although, when compared with the experimental investigation described in Chapter 6 the wastewaters used in this study were collected from a single WWTP in such a way as to minimise the effect of variation in influent SS and SCOD on SCOD removal, it can be seen from Table 7.1 and Appendix D-1 that the characteristics of the wastewater varied greatly during the study period.

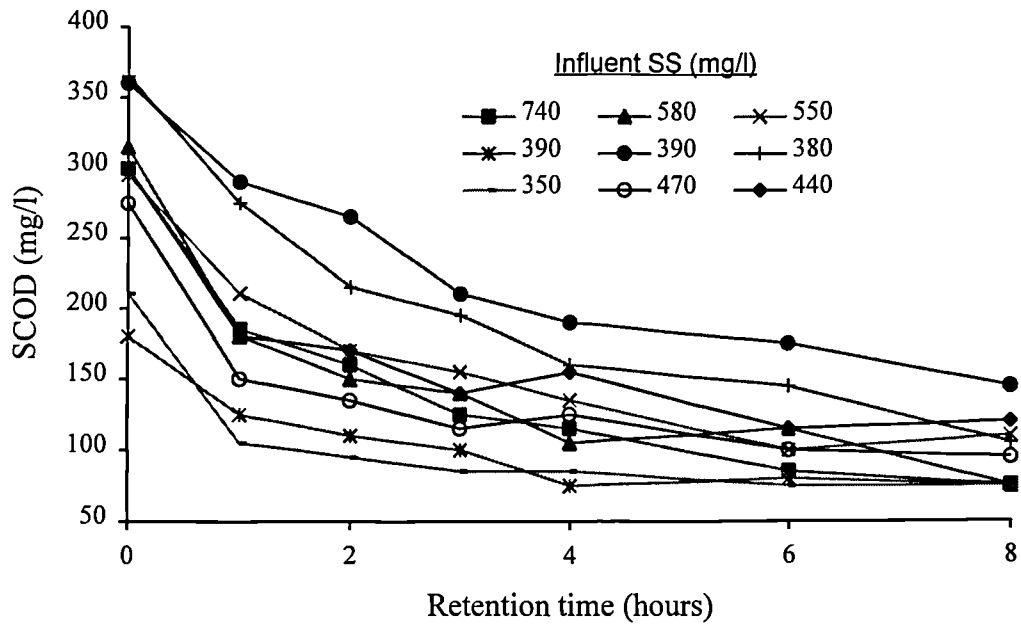
### 7.3. REMOVAL OF SOLUBLE ORGANIC MATTER

Figures 7.1 to 7.5 illustrate the variation in soluble COD (SCOD) of the seeded wastewaters over time. The average SCOD and  $\text{SBOD}_5$  removal efficiencies at different seed concentrations are shown in Tables 7.4 and 7.5.

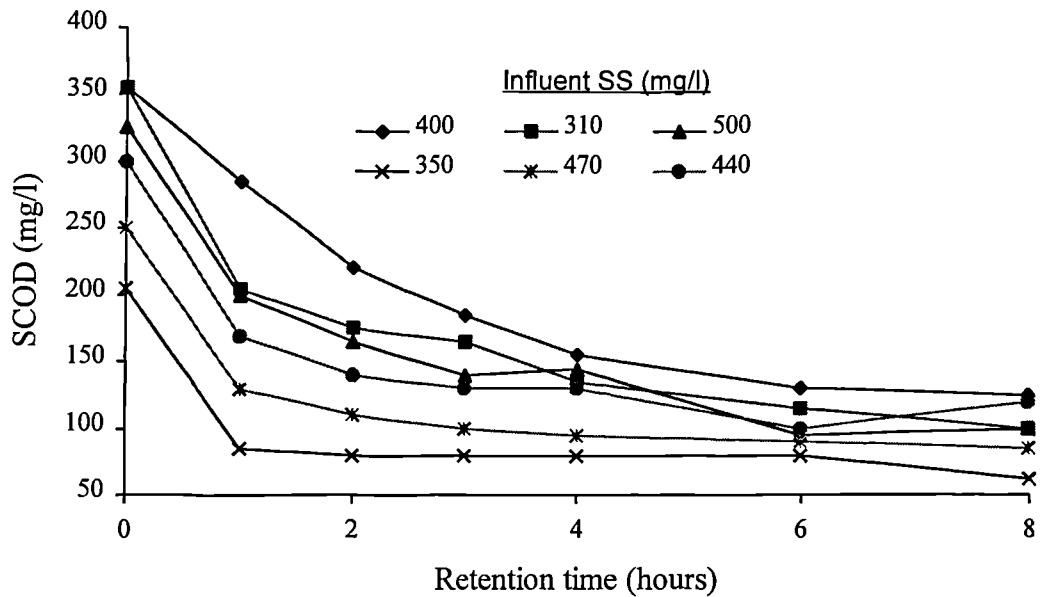
The soluble COD removal efficiencies at different seed concentrations over a retention period of 8 hours and the corresponding “mg of SCOD removed per mg of total VSS (summation of seed concentration and the VSS of the raw wastewater prior to the addition of activated sludge)” are given in Table 7.6.



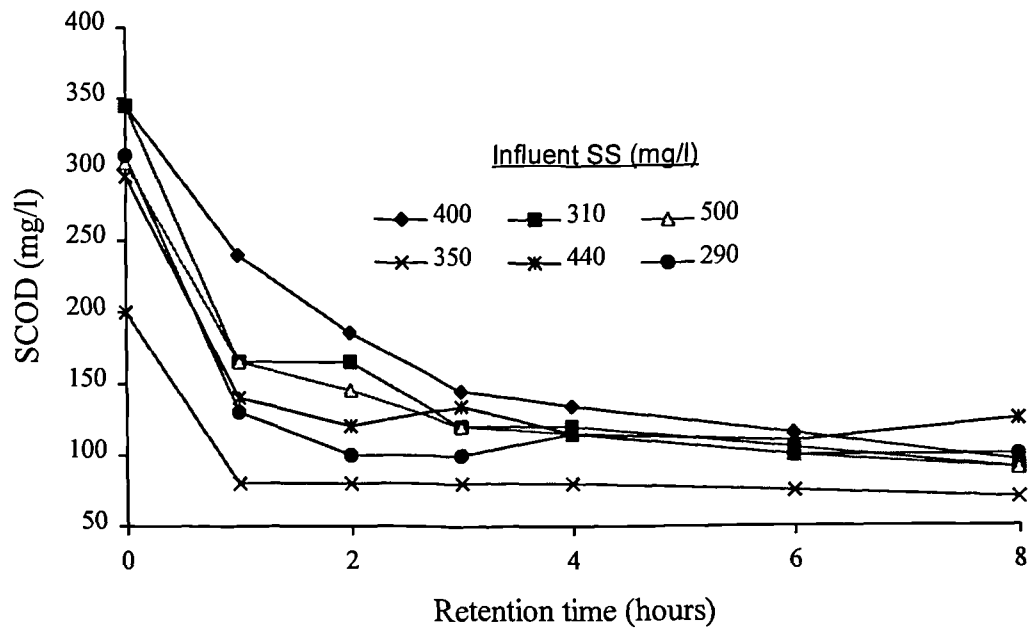
**Figure 7.1-Variation in SCOD of the wastewater at an initial added seed concentration of 100 mg/l at 20°C (Influent SS indicate the SS concentration of the wastewater prior to the addition of activated sludge).**



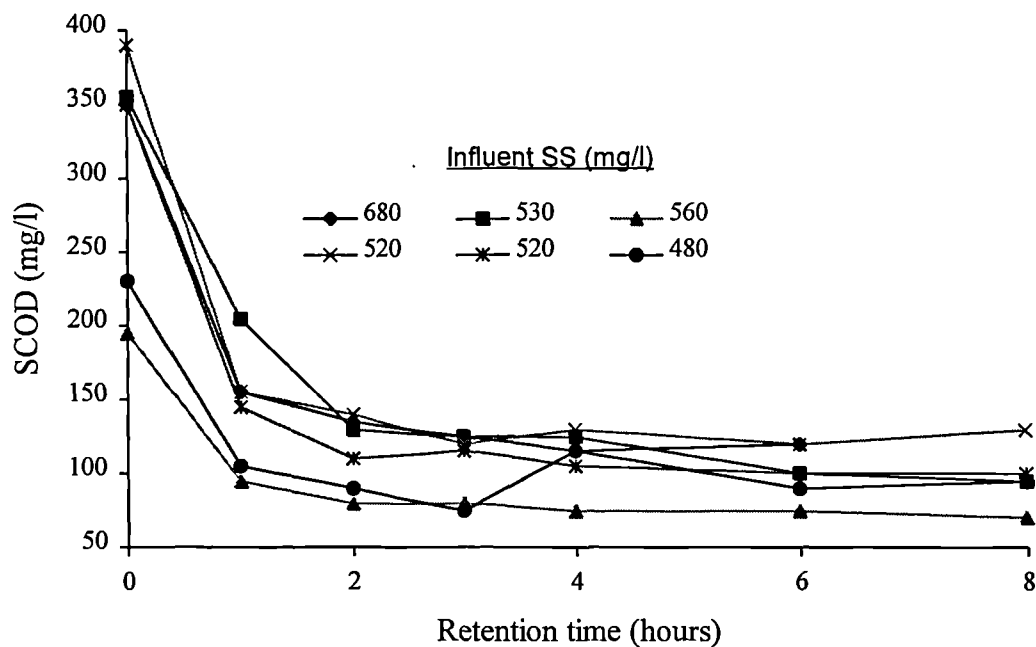
**Figure 7.2-Variation in SCOD of the wastewater at an initial added seed concentration of 250 mg/l at 20°C.**



**Figure 7.3-Variation in SCOD of the wastewater at an initial added seed concentration of 500 mg/l at 20°C.**



**Figure 7.4-Variation in SCOD of the wastewater at an initial added seed concentration of 750 mg/l at 20°C.**



**Figure 7.5-Variation in SCOD of the wastewater at an initial added seed concentration of 1000 mg/l at 20°C.**

**Table 7.4-Average SCOD removal efficiencies (%)**

B t	100	250	500	750	1000
1	25±9	36±10	42±13	50±11	54±6
2	33±10	44±9	51±8	56±8	63±3
3	40±9	52±7	56±5	61±5	65±4
4	45±7	56±6	59±3	62±2	63±5
6	54±9	62±6	66±4	66±3	67±5
8	61±8	66±7	67±4	68±6	67±6

Notes: (i) B=seed concentration (mg VSS/l)

(ii) t=retention time (hours)

**Table 7.5-Average soluble BOD<sub>5</sub> (SBOD<sub>5</sub>) removal efficiencies (%)**

B t	100	250	500	750	1000
2	48±10	62±9	59±10	74±8	75±9
4	65±6	79±6	80±2	92±5	86±5
6	79±6	90±4	89±1	93±3	90±5
8	84±5	92±5	94±3	95±3	94±3

**Table 7.6- Substrate removal efficiencies after a retention period of 8 hours**

Seed concentration	RSCOD-8 (mg/l)	mg RSCOD/mg VSS
100	182±51	0.47±0.12
250	192±50	0.3±0.08
500	200±43	0.23±0.05
750	205±48	0.19±0.05
1000	206±70	0.14±0.05

Note: RSCOD-8=SCOD removal after 8 hours

The food to micro-organisms (F/M) ratio, as computed on the basis of influent total VSS (summation of VSS in the raw wastewater prior to the addition of the seed, and seed concentration), averaged 0.6 (SD=0.2), 0.45 (SD=0.12), 0.35 (SD=0.07), 0.28 (SD=0.05), 0.21 (SD=0.05) g COD/g VSS at seed concentrations of 100, 250, 500, 750 and 1000 mg/l, respectively. The corresponding F/M ratios, when the non-degradable fraction of raw wastewater was neglected, averaged 0.7 (SD=0.01), 0.6 (SD=0.14), 0.42 (SD=0.09), 0.32 (SD=0.06), 0.24 (SD=0.06), respectively. For the above calculations, the non-biodegradable fraction was assumed to be 40% of the VSS of the raw wastewater, as suggested by McKinney (1970).

The effluent SCODs over 8 hours averaged 116 (SD=36), 97 (SD=25), 99 (SD=23) and 98 (SD=21) mg/l at seed concentrations of 100, 250, 500 and 750 mg/l, respectively. The corresponding effluent BOD<sub>5</sub> values averaged 21 (SD=7), 10 (SD=7), 10 (SD=5), 9 (SD=4) and 8 (SD=3) mg/l, respectively.

When the variation in SCODs of the individual wastewaters over time (all the individual curves shown in Figures 7.1-7.5) was described by the first and second order reactions (Appendix C-3), the second order fits yielded higher correlation coefficients for all the wastewaters investigated at seed concentrations of 100 and 250 mg/l. This suggests that the SCOD removal at these seed concentrations, with respect to time, would be best described by second order kinetics.

At 500 mg/l seed concentration, the SCOD removal for all wastewaters but one displayed second order kinetics (Correlation coefficient;  $R^2=0.9-0.97$ ). For a wastewater which had

an initial SCOD ( $SCOD_0$ ) of 205 mg/l, the SCOD removal kinetics were second order during the first 2 hours. The SCOD removal appeared to become asymptotic afterwards as can be observed from Figure 7.3.

At 700 mg/l seed concentration, 50% of the wastewaters investigated showed second order kinetics with regard to SCOD removal over 8 hours. For the remaining 50% wastewaters, the SCOD removal mechanism was second order only over first 2 hours, as very little COD removal was subsequently observed (Figure 7.4).

At a seed concentration of 1000 mg/l, the correlation coefficients ( $R^2$ ) for second order kinetics reaction rates, although higher than the corresponding first order values, were low and varied from 0.2-0.84. The second order model fitting exercise showed that during the first three hours, the SCOD degradation rate mainly followed second order kinetics with the correlation coefficient varying from 0.8-0.92. Beyond 3 hours however, the SCOD removal appeared to level off for most of the wastewaters investigated, and a little SCOD removal was observed thereafter.

In order to empirically describe the changes in SCOD over time in seeded wastewater, regression analysis was carried out on the data shown in Figures 7.1-7.5. The best-fit relationship for the description of SCOD removal over time at an initial added seed concentration of 100 mg/l (all the data shown in Figure 7.1) is as follows:

$$\frac{SCOD_t}{SCOD_0} = \log e \frac{2.65}{(1+t)^{0.27}} \quad (R^2 = 0.84) \quad (7.1)$$



where:  $SCOD_t$  = soluble COD (mg/l) in the seeded wastewater after a retention time  $t$

(1-8 ) hours, and

$SCOD_0$  = soluble COD (mg/l) in the seeded wastewater at the start of the oxygen addition.

The SCOD variation over time at a seed concentration of 250-1000 mg VSS/l (Figures 7.2-7.5) was found to be described by the following best-fit relationship:

$$\frac{SCOD_t}{SCOD_0} = \frac{a}{(1+t)} + b \quad (R^2 = \text{as shown in Table 7.7}) \quad (7.2)$$

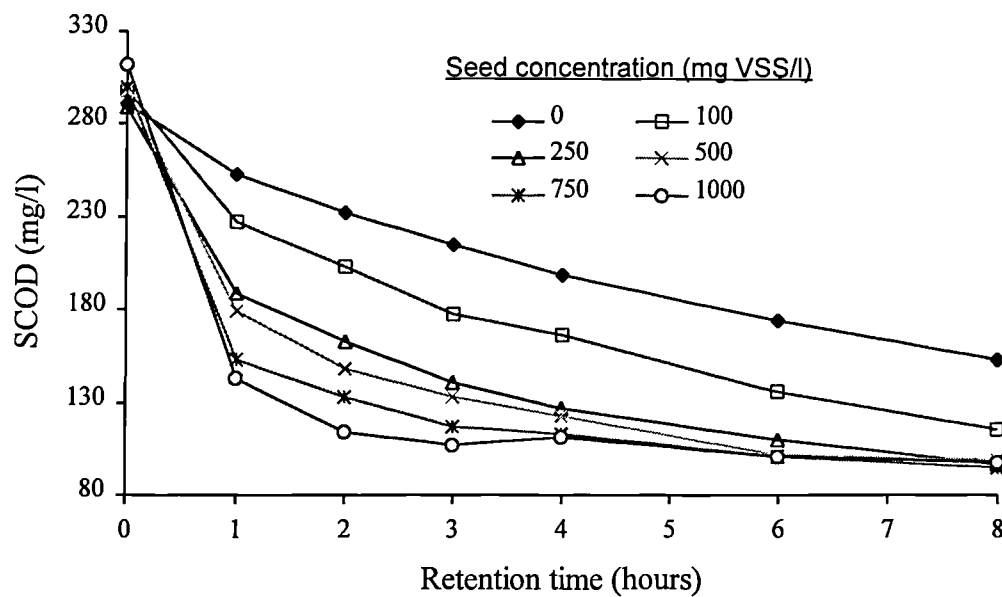
Where  $a$  and  $b$  are constants, with values for individual seed concentrations shown in Table 7.7.

**Table 7.7-Values of constants  $a$  and  $b$  (Equation 7.2) for different seed concentrations**

Seed concentration (mg VSS/l)	$a$	$b$	$R^2$
250	0.72	0.28	0.86
500	0.74	0.24	0.92
750	0.75	0.21	0.91
1000	0.76	0.18	0.89

#### **7.4. EFFECT OF SEED CONCENTRATION ON REMOVAL OF SOLUBLE ORGANIC MATTER**

The average effluent SCODs for different initial seed concentrations are plotted in Figure 7.6. For comparison purposes, the average results of all the unseeded wastewaters (Chapter 6) have also been included.



**Figure 7.6-The average variation in SCOD of seeded and unseeded wastewater.**

From the average results shown in Figure 7.6, it can be seen that an increase in seed concentration from 0-100 and again from 100-250 mg/l led to a marked reduction in SCOD over a retention period of 1 to 8 hours. The effect of increasing the seed concentration beyond 250 mg/l on SCOD removal however, appears to be dictated by the retention period, and seems to be noticeable only when the retention period was about 3 hours.

In efforts to further delineate the effect of seed concentration on SCOD removal while taking into account the variation in SCODs and SS concentrations of the individual wastewaters prior to the addition of activated sludge (Table 7.1), multiple regression analysis was used. For this purpose, the SCOD removal efficiencies (RSCOD) over a retention period of 1 to 8 hours for all the wastewaters investigated at seed concentrations of 100-1000 mg/l (36 wastewaters) and those studied without any seed concentration (26

wastewaters; Chapter 6), were analysed together with their corresponding  $SS_0$  and  $SCOD_0$  values. Appendix D-1 shows how the above data were arranged so that multiple regression analysis could be carried out. The empirical models for SCOD removal efficiencies over a retention period of 1 to 8 hours, obtained from multiple regression analysis, are shown in Table 7.8.

**Table 7.8-Empirical equations showing the effect of seed concentration,  $SCOD_0$  and  $SS_0$  on SCOD removal** (seed concentration = 0-1000 mg/l)

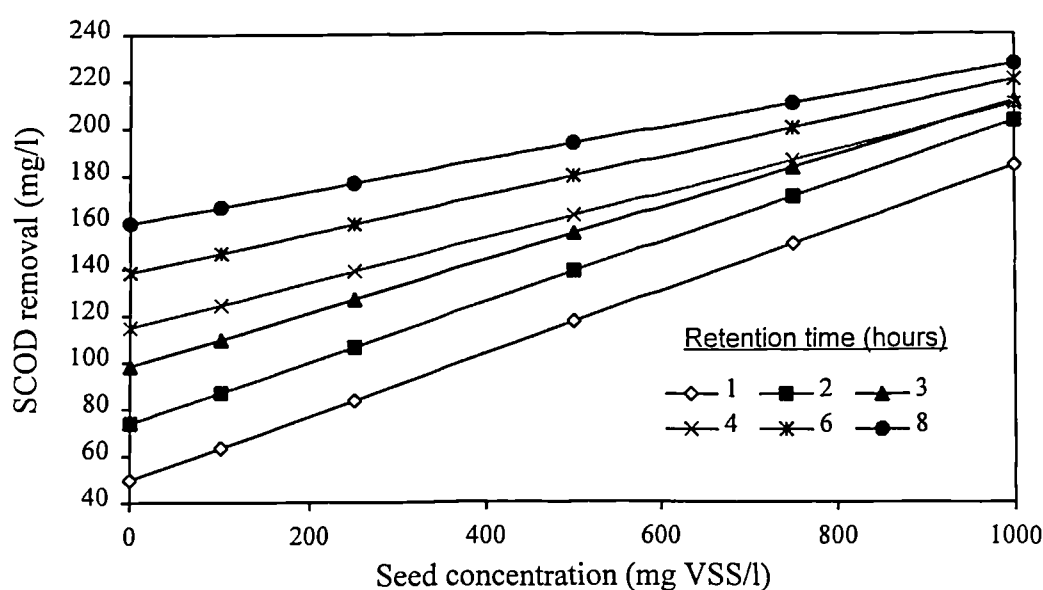
Retention time (hours)	RSCOD (mg/l)	Correlation coefficient ( $R^2$ )
1	$-11.8 + 0.093 SS_0 + 0.0655 SCOD_0 + 0.134 B$	0.80
2	$-31.8 + 0.104 SS_0 + 0.196 SCOD_0 + 0.129 B$	0.81
3	$-43.4 + 0.11 SS_0 + 0.307 SCOD_0 + 0.113 B$	0.81
4	$-52.9 + 0.0999 SS_0 + 0.409 SCOD_0 + 0.095 B$	0.81
6	$-52 + 0.0914 SS_0 + 0.497 SCOD_0 + 0.0821 B$	0.82
8	$-61.7 + 0.123 SS_0 + 0.553 SCOD_0 + 0.0675 B$	0.81

Notes: (i)  $SS_0$  = SS concentration in the raw wastewater (mg/l) prior to the addition of activated sludge  
(ii)  $SCOD_0$  = soluble COD in the reactor at the start of the experiment (mg/l)  
(iii) B = seed concentration added to the reactor at the start of the experiment (mg VSS/l)  
(iv) RSCOD = SCOD removal (mg/l)

From the equations presented in Table 7.8, it can be observed that fairly high correlation coefficients ( $R^2 \geq 0.8$ ) were obtained. This indicates that the operating variables selected for the prediction of the response i.e. RSCOD are adequate, and the above equations (Table 7.8) could be used to predict the effect of seed concentration on SCOD removal efficiency.

The empirical models shown in Table 7.8 were used to deduce the SCOD removal efficiencies, at different seed concentrations, for a wastewater with  $SCOD_0$  and  $SS_0$

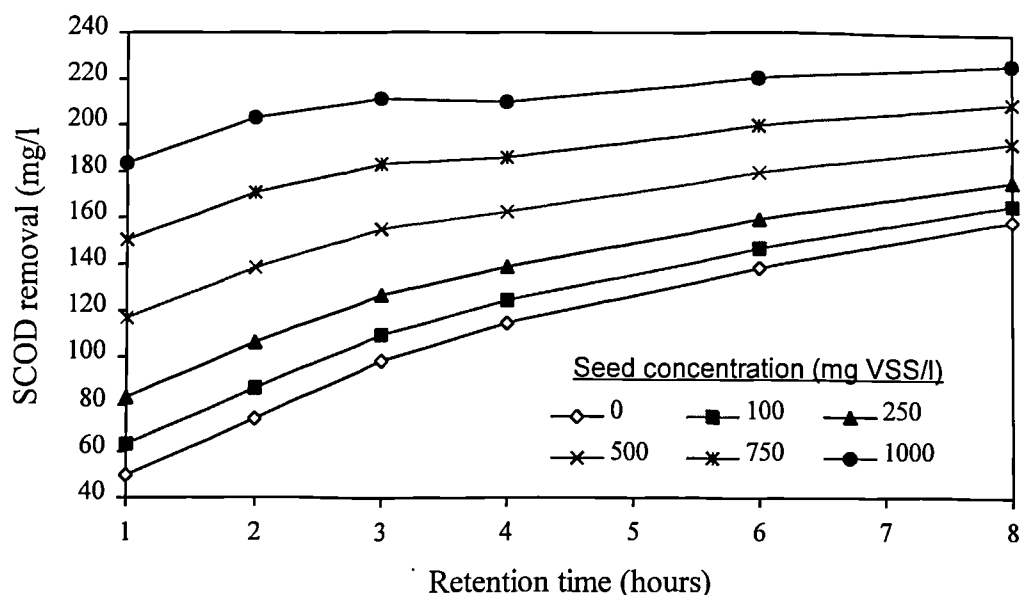
concentrations of 300 and 450 mg/l, respectively. These values of  $SS_0$  and  $SCOD_0$  were assumed considering the average  $SCOD_0$  and  $SS_0$  concentrations of 300 and 450 mg/l respectively, for 36 individual wastewaters investigated at a seed concentration of 100-1000 mg/l. Based on the results of the empirical models (Table 7.8), theoretical curves shown in Figure 7.7 were generated to illustrate how the seed concentration would affect the SCOD removal at constant  $SCOD_0$  and  $SS_0$  concentrations.



**Figure 7.7-Effect of seed (activated sludge) concentration on SCOD removal.**

Based on the data shown in Figure 7.7, it appears that the SCOD removal in seeded wastewater under aerobic conditions increases linearly with an increase in the seed concentration in the range of 100-1000 mg/l. The effect of seed concentration on SCOD removal however, appears to diminish with an increase in the retention period, as indicated by a decrease in the slope of lines shown in Figure 7.7.

Figure 7.8 shows the results presented in Figure 7.7 but re-plotted to show more clearly the effect of retention time on SCOD removal at individual seed concentrations. The results shown in Figure 7.8 reveal that the SCOD removal rate is at a maximum during the first four hours for seed concentrations of 100-500 mg/l, and first two 3 hours for higher seed concentrations. Thereafter, the SCOD removal rate is relatively low and appears to become constant.



**Figure 7.8-Effect of retention time on SCOD removal/**

When multiple regression analysis was carried out on all the SCOD data shown in Figures 7.1-7.5, to statistically determine the effect of  $SCOD_0$ ,  $SS_0$ , retention time and seed concentration on SCOD removal efficiency, the following empirical equation was obtained:

$$\text{RSCOD} = -110 + 0.479 \text{SCOD}_0 + 0.0457 \text{SS}_0 + 17.2 t + 11.2 b - 1.11 bt \quad (R^2 = 0.77) \quad (7.3)$$

Where: RSCOD = SCOD removal (mg/l) after a retention time  $t$  (1-8) hours from the start of the experiment, and

$$b = \{1 + \text{seed concentration in mg per litre}/100\}.$$

The summary statistics of Equation 7.3, including the estimated coefficients for the independent variables involved, their standard deviations and  $t$ -ratios, is shown in Appendix D-2. An inspection of the summary statistics of the regression analysis revealed that at a significance level of 5%, the SCOD removal in the seeded wastewater was affected significantly by  $\text{SCOD}_0$ ,  $\text{SS}_0$ , retention time and seed concentration (in the range of 100-1000 mg/l). The effect of  $\text{SS}_0$  however, was found to be least significant. Furthermore, the effect of interaction between seed concentration and retention time was also found to be significant.

## 7.5. EFFECT OF OXYGEN ADDITION ON BIOMASS

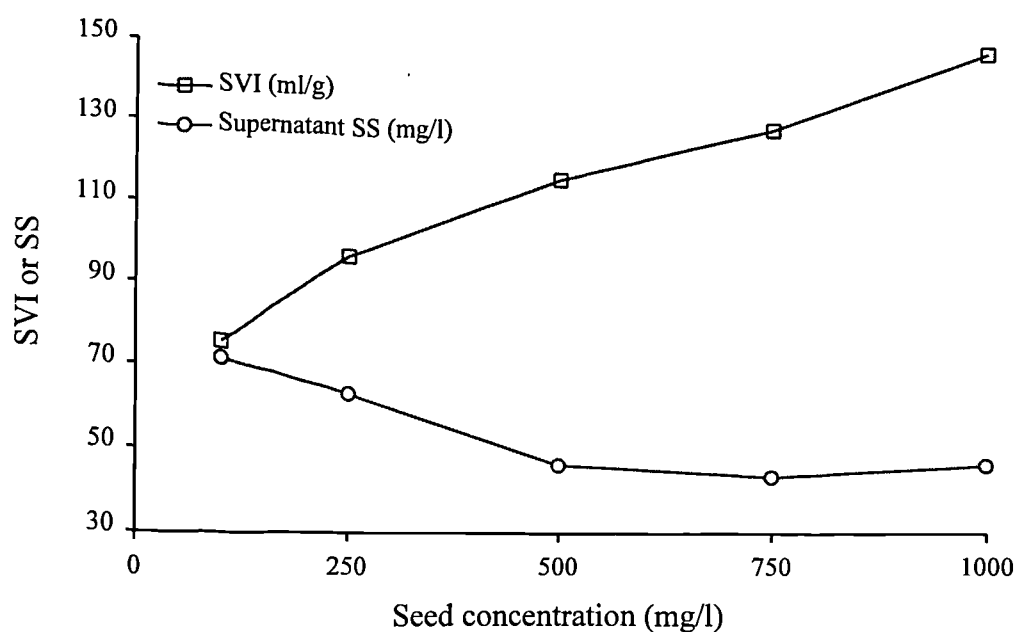
To estimate the changes in the VSS (biomass) of the seeded wastewater (mixed liquor) as a result of oxygen addition, the VSS of the mixed liquor were measured before, and after six hours of oxygen addition in the batch reactor.

The VSS of the mixed liquor at all seed concentrations investigated, in general, showed a decrease over 6 hours although a few wastewaters also showed a slight increase. The overall change in mixed liquor volatile suspended solids (MLVSS) after six hours of oxygen addition ranged from -12 to +23, -10 to 5, -18 to 9, -13 to 2, and -6 to -16%,

respectively, at seed concentration of 100, 250, 500, 750 and 1000 mg/l. The MLVSS results also showed that the decrease in MLVSS tended to increase with an increase in the seed concentration.

## 7.6. EFFLUENT SETTLEABILITY

To evaluate the settling characteristics of the solids in the effluent at different seed concentrations, a sample of mixed liquor after a retention period of 6 hours was taken from the reactor and subjected to a SVI test. A sample of the mixed liquor was collected from the batch reactor at the same time and settled in a bench-scale settling column for one hour and the SS of the supernatant was determined. The results of SVI tests and the SS of the supernatant from the settling column are plotted in Figure 7.9 and summarised in Table 7.9.



**Figure 7.9-Effect of seed concentration on SVI and SS of the supernatant.**

**Table 7.9- Settleability test results**

Seed concentration	SVI (ml/g)	Supernatant SS (mg/l)
100	76±10* (9)	72±15 (5)
250	98±13 (9)	64±21 (5)
500	118±19 (6)	47±12 (6)
750	130±21 (6)	44±16 (6)
1000	149±13 (6)	47±13 (6)

Notes: (i) \* average±SD

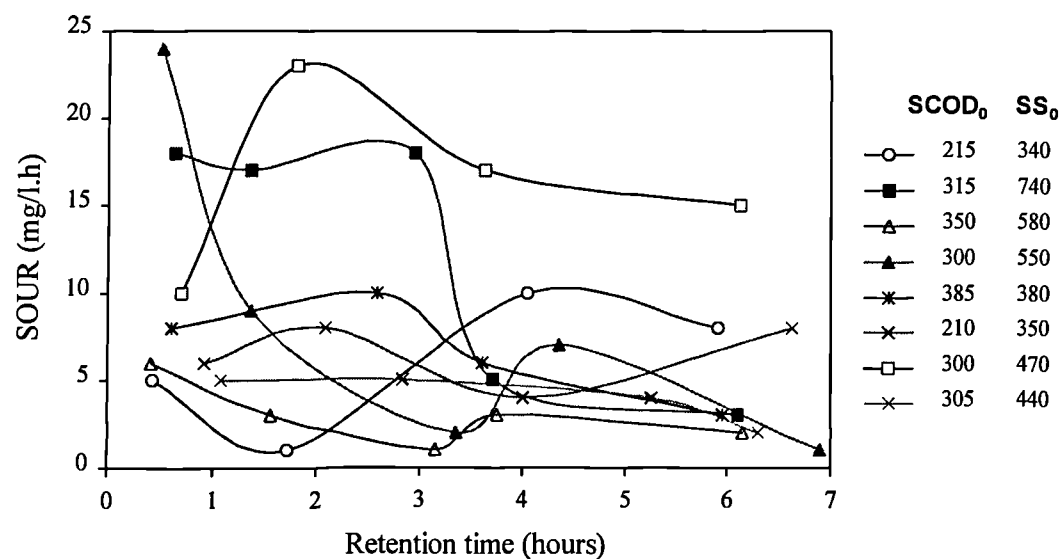
(ii) the values in parenthesis indicate the number of experimental runs

The results shown in Table 7.9 and Figure 7.9 indicate that the SVI increases steadily with an increase in seed concentration up to 1000 mg/l. The SS of the effluent from the settling column however, appear to decrease gradually with an increase in seed concentration up to 500 mg/l. Beyond 500 mg/l, the effluent SS appears to be independent of the seed concentration, as can be seen from Figure 7.9.

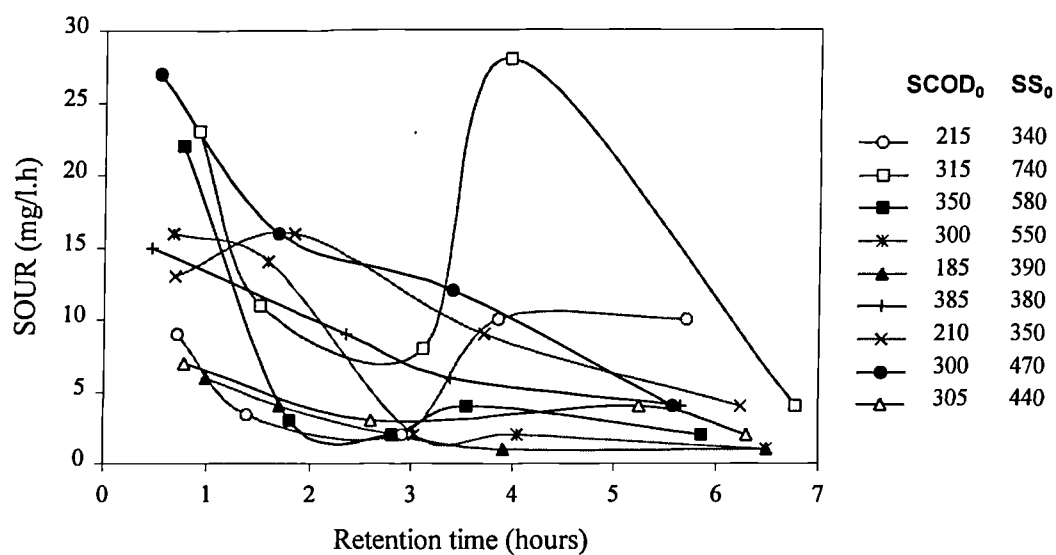
## 7.7. OXYGEN UPTAKE RATE

To estimate the variations in oxygen uptake rate of the seeded wastewater (SOUR), samples of the oxygenated mixed liquor were taken from the batch reactor at specified intervals of time and the SOUR was determined as described in Section 4.3.1. Figures 7.10-7.14 illustrate the variation in SOUR of the wastewater at seed concentrations of 100-1000 mg/l, at 20°C.

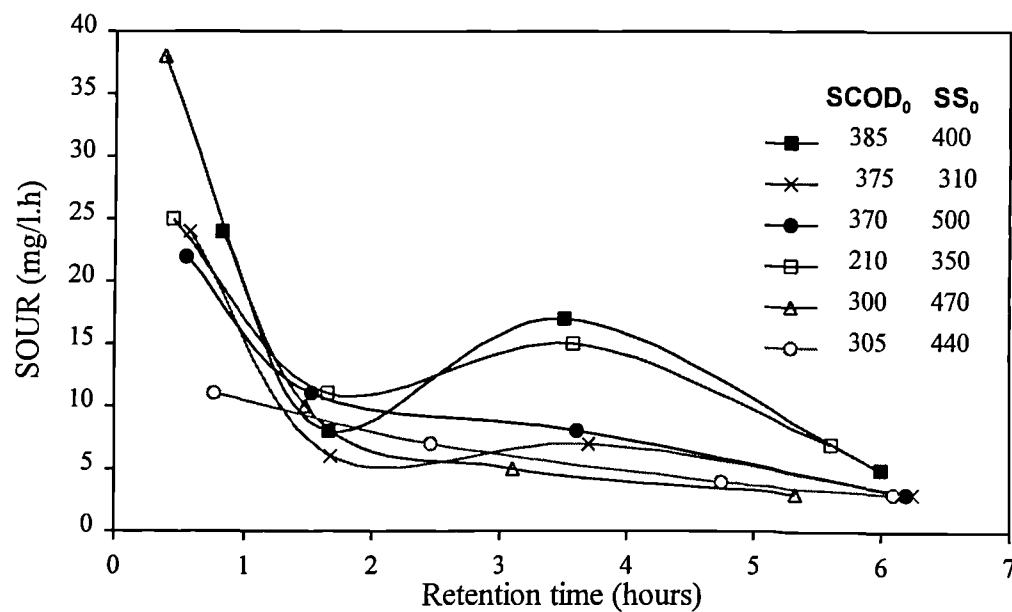




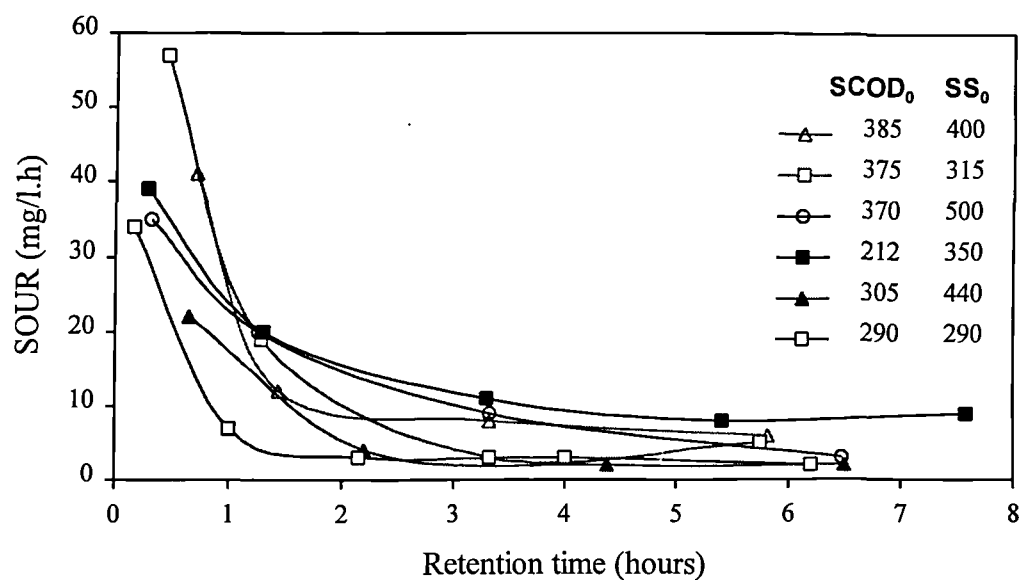
**Figure 7.10-SOUR profile of the wastewater at an initial added seed concentration of 100 mg/l (SCOD<sub>0</sub> and SS<sub>0</sub> are expressed in mg/l).**



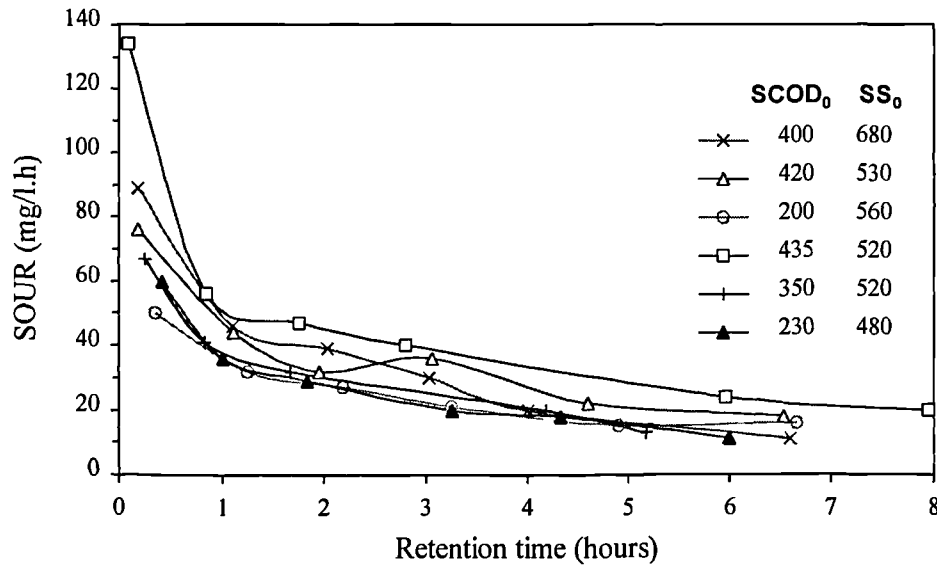
**Figure 7.11-SOUR profile of the wastewater at an initial added seed concentration of 250 mg/l.**



**Figure 7.12-SOUR profile of the wastewater at an initial added seed concentration of 500 mg/l.**



**Figure 7.13-SOUR profile of the wastewater at an initial added seed concentration of 750 mg/l.**



**Figure 7.14-SOUR profile of the wastewater at an initial added seed concentration of 1000 mg/l.**

To evaluate the effect of SS of the wastewater prior to the addition of activated sludge, the initial SCOD in the reactor, and the retention time, on the SOUR at different seed concentrations, multiple regression analysis was performed on the data shown in Figures 7.10-7.14. The empirical equations describing the SOUR of the seeded wastewater over time, and their summary statistics are shown in Appendix D-3.

From the empirical equations presented in Appendix D-3 and from their summary statistics, it was inferred that:

- (i) at a seed concentration of 100 mg/l, the SOUR of the wastewater did not appear to show any significant correlation with  $SS_0$ ,  $SCOD_0$  or the retention time. Also, none of the above parameters were found to have a significant influence on SOUR.
- (ii) at a seed concentration of 250, 500 and 750 mg/l, only the effect of retention time on SOUR was found to be significant, and

- (iii) at a seed concentration of 1000 mg/l, the SOUR was significantly affected both by  $\text{SCOD}_0$  and retention time.

## 7.8. DISCUSSION

### 7.8.1. Removal of soluble organic matter

The results of the present study demonstrate that, when compared with the soluble organic matter removal in the unseeded wastewater (Chapter 6), the addition of activated sludge to the wastewater, at a concentration as low as 100 mg VSS/l, would result in a considerable increase in soluble organic matter removal and hence a significant reduction in the time required to achieve a given degree of in-sewer treatment. For unseeded wastewaters (Chapter 6), the SCOD and  $\text{SBOD}_5$  removal efficiencies (for 24 wastewaters) over a retention period of 8 hours averaged 48% and 66%, respectively. When compared to this, the SCOD and  $\text{SBOD}_5$  removal efficiencies at a seed concentrations of 100 mg VSS/l averaged 54 and 79% respectively, over a retention period of 6 hours (Tables 7.4 and 7.5). The average SCOD and  $\text{SBOD}_5$  removal efficiencies at a seed concentration of 250 mg VSS/l amounted to 56 and 79%, respectively, over a retention period of 4 hours.

The results shown in Tables 7.4 and 7.5, and Figures 7.6-7.8 illustrate that the effect of an increase in seed concentration on substrate removal efficiency tends to diminish with the increase in retention time. Moreover, the improvement in SCOD removal efficiency as a result of increasing the seed concentration was not in proportion to the concentration of seed added. Over a retention period of 8 hours, for instance, an increase in seed concentration of 300% (250 to 1000 mg/l), increased the SCOD removal (predicted from

Table 7.8) by 66, 34, 25 and 19%, when the retention period was 2, 4, 6 and 8 hours, respectively. Similarly, increasing the seed concentration from 750 to 1000 mg/l resulted in an increase of 15, 9, 7 and 5% in SCOD removal, over a retention period of 2, 4, 6 and 8 hours, respectively.

The results of the present study with respect to the effect of seed concentration on SCOD removal at higher retention periods support the findings of Green *et al.* (1985) who, based on laboratory studies undertaken to simulate a step-fed sewer having a retention period of 10 hours, reported that decreasing the initial seed concentration by about 70% resulted in only a small decrease (6%) in the dissolved COD removal.

A diminished effect of seed concentration on substrate removal efficiency with an increase in retention time may be due to the fact that at higher seed concentrations, more active biomass and more substrate is present initially, thus resulting in a considerably higher SCOD removal during the first few hours. The removal rate subsequently decreases as soon as the limiting substrate conditions are reached and thus little removal is observed with an increase in retention period.

A relatively small improvement in SCOD removal efficiency with an increase in seed concentration in the range of 250-1000 mg/l, at higher retention periods (Table 7.4 and Figures 7.6-7.8) could also be explained by the sludge activity theory advanced by Tench (1968, 1994). According to Tench (1994) the rate of oxidation of wastewater is proportional to the number of viable micro-organisms in the activated sludge and the concentration of the substance adsorbed on the floc surface. He, based on experimental

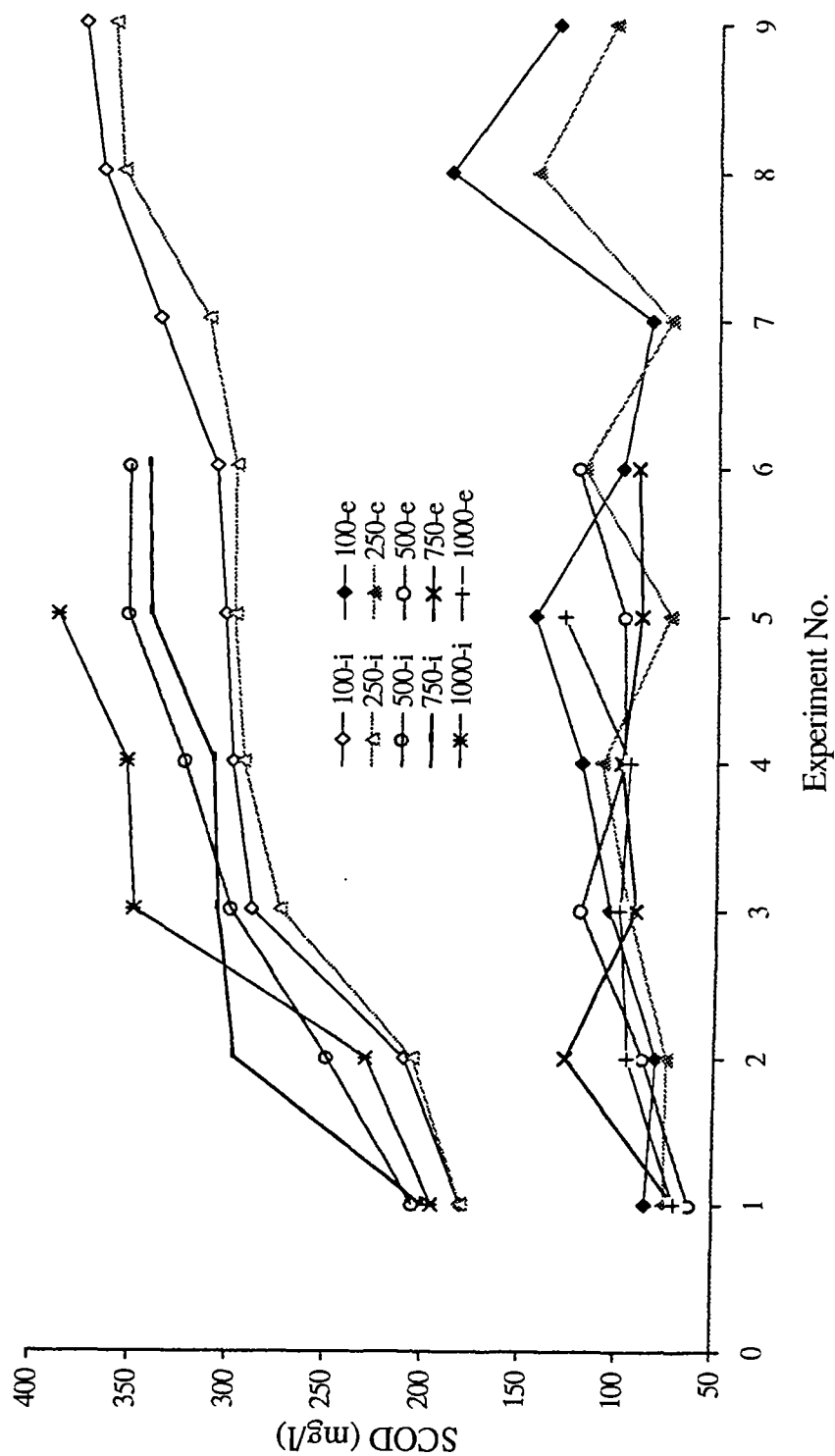
trials with full-scale activated sludge plants, noted that “the active biomass is a decreasing proportion of the sludge as its concentration is increased”. Tench (1994) also suggested that there is an optimum sludge concentration for each plant and that the sludge could be varied greatly without significantly affecting the plant treatment efficiency. An increase in MLSS beyond the optimum concentration caused only a small reduction in effluent quality. Based on full-scale experimental results, Tench (1994) developed an empirical equation linking sludge viability and its concentration, which he used to demonstrate that a 100% increase in MLSS in the aeration tank would result in only about 10% increase in the active biomass concentration of the sludge.

From the results shown in Tables 7.4 and 7.5, it can be seen that the soluble BOD<sub>5</sub> removal efficiencies in excess of 90% were achieved over 8 hours, at seed concentrations of 250-1000 mg/l. The BOD<sub>5</sub> removal efficiency in a conventional activated sludge plant (ASP) which is usually designed to operate at a mixed liquor suspended solids concentration of 1200-3500 mg/l, ranges from 85-95% (Horan, 1991). Considering the above, the substrate removal efficiencies found in the current study are quite comparable to those found in a typical ASP, inspite of considerably lower MLSS concentrations than those found in conventional ASPs. According to Grutsch and Mallatt (1976) as quoted by Capps *et al.* (1995), the critical MLSS concentration, the concentration of MLSS below which the activated sludge treatment becomes difficult, is about 700 mg/l. Shao *et al.* (1992) have reported an excellent effluent quality from a high-rate activated sludge plant employing a MLSS concentration of 950 mg/l. From the results shown in Tables 7.4 and 7.5, it appears that, within the range of operating parameters investigated in the present study, a sewer with a seed concentration of about 500-1000 mg/l being added at its inlet may prove an

alternative to a conventional aeration tank. The optimum concentration of seed to be added to an in-sewer treatment system to achieve a given degree of treatment however, would depend upon the strength of the wastewater, the retention time available, temperature, and the characteristics of the sewerage system.

High substrate removal efficiencies at low biomass concentrations, as is the case in the present study (Table 7.6), could also be explained by the sludge activity theory as described by Green and Shelef (1980). According to them, the sludge activity in a conventional activated sludge plant is low (less than 20%), because of the low net growth caused by the limiting substrate conditions in the reactor. On the other hand, an increase in the food to micro-organism ratio ( $> 0.3$  per day) causes higher net growth which results in steadily increasing sludge viability. This high sludge viability therefore, compensates for low MLVSS concentration, thus resulting in a high substrate removal.

According to Moser *et al.* (1977), tubular (pipe) reactors are likely to exhibit some instability against step changes in inflow or concentration, which could be overcome by the control of return sludge depending upon the  $BOD_5$  concentration of the influent. In the present study, the effluent SCOD at seed concentrations of 100 mg/l was found to fluctuate with variation in influent concentrations similar to that observed for unseeded wastewaters (Chapter 6). This instability in effluent concentration however, appeared to dampen out with an increase in seed concentration as shown in Figure 7.15.



**Figure 7.15-Effect of influent SCOD on the effluent SCOD concentrations.**

Notes: numbers shown in legend represent seed concentration e.g.

(a) 100-i = influent SCOD at a seed concentration of 100 mg VSS/l

(b) 100-e = effluent SCOD (after a retention period of 8 hours) at a seed concentration of 100 mg VSS/l



Most of the studies undertaken to investigate the SCOD or BOD<sub>5</sub> removal kinetics in wastewater treatment report that the substrate removal in an activated sludge plant follows first order kinetics (Wolfbauer *et al.*, 1978; Newbry *et al.*, 1988). Wolfbauer *et al.* (1978), on the other hand, demonstrated that the process follows a zero order reaction. Contrary to the finding of the above studies, the SCOD removal rate in the seeded wastewaters investigated in the present study appeared to follow mainly second order kinetics. This observation however, is in accordance with the results of Tucek and Chudoba (1969), who, based on laboratory and field work, concluded that the course of BOD<sub>5</sub> removal in batch or plug flow systems could be described by a second order reaction rate.

All the wastewaters studied at a seed concentration of 100-1000 mg/l showed a maximum SCOD removal during the first hour of oxygen addition. The SCOD removal over a retention period of 1 hour averaged 25, 36, 42, 50 and 54% (Table 7.4) at a seed concentration of 100, 250, 500, 750 and 1000 mg/l, respectively. The corresponding values at a retention period of 8 hours were found to be 61, 66, 67, 68, 67%, respectively. A high SCOD removal rate, initially, may be due to the oxidation of readily biodegradable material (Braha and Hafner, 1986), and simultaneous physical adsorption of particulate organics and biochemical absorption of soluble organics (Rickert and Hunter, 1971).

The SCOD removal for wastewaters studied at a seed concentration of 1000 mg/l was found to approach nearly a stationary value after a retention period of 2-4 hours (Figure 7.5). The SCOD removal averaged 63 and 67% over a retention period of 2 and 8 hours, respectively. These observations are consistent with those of Rickert and Hunter (1971),

who, while investigating the effect of aeration time on SCOD removal in a plug-flow aeration tank, showed that soluble COD removal reached nearly a stable level after one hour of aeration. They found that the soluble COD removal averaged 77% over 1 hour and 78% over 6 hours. Stoyer and Scherfig (1972), based on a theoretical quantitative comparison of completely mixed and plug flow reactors, demonstrated that the theoretical time required to achieve a given degree of treatment in a plug flow reactor is only about one-third of that required in a completely mixed system. They, based on pilot-scale tests involving the addition of activated sludge to a pressure pipe, showed that most of the organic matter in a plug-flow in-pipe treatment system was removed in about 1 hour.

From the SCOD profiles shown in Figure 7.5, it can be seen that at a seed concentration of 1000 mg/l, the SCOD reduces up to 3 hours, then shows an increase and is again reduced to a stationary value. This phenomenon may be attributed to the generation of soluble microbial products (Germali *et al.*, 1991). According to Boero *et al.* (1991), a major fraction of the soluble organic carbon in biological wastewater effluents consists of soluble microbial products, which may be either substrate utilisation associated products (those resulting from intermediates or end products of substrate degradation, cell metabolism, or cell growth) or biomass associated products which result from cell lysis and decay.

### **7.8.2. Biomass growth**

The change in biomass in the seeded wastewater, measured as the change in MLVSS after a retention period of 6 hours (Section 7.5), varied widely and appeared to decrease with respect to the MLVSS concentration in the reactor at the start of oxygen addition.

Considering the fact that a significant degree of SCOD removal occurred over 6 hours at all the seed concentrations investigated (Table 7.4), a decrease in VSS as a result of oxygen addition was unexpected. The results of the current study with regard to biomass growth are not consistent with the sludge viability theories cited in previous section. These results are also contrary to the findings of Green *et al.* (1985), who reported 3.1 and 5.2 fold increases in sludge viability, in a simulated step-fed plug flow sewer at initial seed concentrations of 4760 and 1430 mg VSS/l, respectively.

As regards the changes in VSS as a result of oxygen addition, the unseeded wastewaters (Section 6.7.3) exhibited a trend similar to that observed in the current study. The possible explanations for this phenomenon were outlined in Section 6.7.3. In addition to the reasons stated in Section 6.7.3, a probable reason for the decrease in VSS observed in the present study may be related to the use of VSS for the measurement of biomass. According to Benfield and Randall (1980) the VSS measurement for biomass “is suspect” and, “inconsistencies do arise when VSS is used as a measure of active biomass when the raw wastewater contains a large fraction of non-biological suspended solids material”, as was the case in the current study.

It should be noted that the substrate concentrations used in the present study (average SCOD=300 mg/l) were considerably lower than the substrate concentrations used by Green *et al.* (1985) (SCOD=590-634 mg/l). Moreover, they used synthetic wastewater, prepared by the addition of yeast to settled domestic wastewater, which may not truly simulate the behaviour of the raw wastewater flowing in sewers. Also, the sludge viability

was measured using substrate removal and oxygen utilisation rates (Green and Shelef, 1981).

A decrease in VSS observed in the present study may also be associated with the way in which the sludge was handled before it was added to the wastewater. The sludge used in the current study originated from a CFSTR activated sludge plant, and was kept under anaerobic conditions for about 1 hour during transportation to the laboratory, and for another one hour during which it was allowed to settle to increase the SS concentration. Due to the length of time required to undertake an experiment, it was not possible to use the sludge on the same day. The sludge was therefore, kept aerated over night without any supply of substrate.

Chudoba *et al.* (1991) investigated the effect of anaerobic stabilisation of returned activated sludge in a continuous system on biomass production under batch conditions at various F/M ratios. It was found that at low  $S_0/X_0$  (substrate/biomass ratios) cell replication did not take place during substrate removal. At higher  $S_0/X_0$  ratios ( $>2$ ) however, a significant increase in biomass concentration was observed. They noted that under anaerobic conditions (for 3 hours) in the absence of substrate, micro-organisms are subjected to physiological shock caused by the lack of oxygen and food. Under the above conditions, ATP is used as a source of energy. When the micro-organisms are returned to aerobic conditions and supplied with exogenous substrate, they rebuild energy reserves at the expense of growth. The anaerobic conditions however, did not effect the system's assimilative capacity.

### 7.8.3. Effluent settleability results

In the current study, SVI (sludge volume index) rather than SSVI (stirred sludge volume index) was used to evaluate the settling characteristics of the mixed liquor because of the fact that the mixed liquor suspension was too dilute, especially at low seed concentrations, to form an interface in a stirred cylinder. SVI however, is widely used as an operational parameter (Echeverria *et al.*, 1992). A detailed discussion about the comparison of the working ranges of SVI and SSVI for the range of solids concentrations normally occurring in activated sludge plants has been presented by Rachwal *et al.* (1982). Although there seems to be no information available in the literature about the comparison between quiescent and stirred tests for the MLSS ranges used in the current study, Vesilind (1971), based on laboratory studies undertaken to evaluate the effect of stirring on settling velocity of activated sludges noted that at relatively dilute concentrations, stirring seemed to have no beneficial effect on settling velocity, irrespective of the size of cylinder.

Activated sludge exhibits a good settleability if the SVI is less than 100, while a SVI value of greater than 150 has been reported to indicate settleability problems and possible bulking (Gray, 1989). Considering the above values of SVI, it can be observed from Table 7.9 that the average SVI values found in the present study, at seed concentration of 100-1000 mg/l, are lower than 150 ml/g thus representing satisfactory settleability characteristics.

As can be seen from Table 7.9 and Figure 7.9, the settleability (SVI) tends to shift from “moderate” to “poor” (Gray, 1989) as the seed concentration increased from 100-1000

mg/l (average F/M= 0.6-0.21 g COD/g MLVSS, Section 7.3). This is in accordance with the findings of Echeverria *et al.* (1992) who, based on pilot-scale studies reported that SVI increases with a decrease in F/M ratio in the range of 0.41-0.1 g BOD<sub>5</sub>/g MLVSS. In contrast, Grutsch and Mallatt (1976) as quoted by Capps *et al.* (1995) suggest that lower MLSS concentrations (less than 700 mg/l) in activated sludge plant result in poor sludge settling and therefore poor clarifier performance. Shao *et al.* (1992) based on comparative studies on full-scale activated sludge plants (ASP) reported that the SVI of high rate ASP (MLSS=900 mg/l) was higher than the conventional ASP (210 ml/g vs. 147 ml/g). Pilot-scale studies on a pressure pipe treatment system at Irvine Ranch Water District California (Section 2.2) also encountered significant problems with solids separation using flotation (Scherfig, 1994).

A probable reason for the increase in SVI with an increase in seed concentration observed in the current study may be the addition of starved sludge to the wastewater for seeding purposes. As mentioned earlier (Section 7.8.2), the seed sludge was kept aerated overnight without feeding any substrate. Horan and Shanmugan (1986), who investigated the effect of nutrient starvation on the settling properties of activated sludge, found that nutrient starvation resulted in a loss of settleability. They attributed this effect to extensive cell lysis which caused pin flocs, and possible high growth rate of filamentous organisms at low nutrient concentrations.

From Table 7.9 and Figure 7.9, it can be observed that inspite of low SVIs at seed concentrations of 100 and 250 mg/l, the suspended solids removal in the bench-scale settling column was lower than those found at higher seed concentrations. From these

results it appears that suspended solids removal at these seed concentrations may be poor, possibly due to lack of adequate bioflocculation because of low biomass concentration.

#### **7.8.4. Oxygen uptake rate**

The results presented in Figures 7.10 and 7.11 show that the SOUR at seed concentrations of 100 and 250 mg/l varied widely and did not show any specific trend. Also the SOUR values at these seed concentrations correlated poorly with SS of the wastewater prior to the addition of activated sludge, initial SCOD, and the retention time. SOUR profiles similar to the above were also observed for unseeded wastewaters (Section 6.6), and a number of possible explanations for the variation in SOUR have been outlined in Section 6.7.4.

The results shown in Figures 7.12-7.14 indicate that, when compared with unseeded wastewater (Section 6.6), the SOUR of the seeded wastewater (at a seed concentration of >250 mg/l) would, in general, be at a maximum following the addition of seed to the wastewater and then would decrease with time. A similar oxygen profile has been reported for a plug-flow activated sludge process by other researchers. Kesseler and Nicholas (1935) reported SOUR values of up to 137 mg/l.h at the inlet and 28 mg/l.h after 2.5 hours of aeration. Randall *et al.* (1991), while investigating the SOUR of raw wastewater added with activated sludge in a batch reactor, noted a peak SOUR of 15 mg O<sub>2</sub>/g MLSS.h which decreased to 7 mg O<sub>2</sub>/g MLSS.h after a retention time of 6 hours.

A sharp increase in SOUR following the addition of seed to the wastewater may be attributed to the rapid utilisation of readily biodegradable organic matter (RBOM). The SOUR decreases as the RBOM is depleted, and is subsequently dominated by growth on

substrate released by hydrolysis and less by endogenous respiration (Kappeler and Gujer, 1992).

The results presented in Figures 7.12-7.14 show that the initial SOUR of the wastewater increases with an increase in seed concentration. This is in agreement with the results reported in Water Pollution Research Report (1964). Schulze and Kooistra (1969) quoting Water Pollution Research Report (1964) reported inlet SOUR values of 70, 100, and 140 mg/l.h for domestic wastewater at MLSS concentration of 2000, 4000, and 6000 mg/l, respectively. The corresponding values at the end of the aeration tank were found to be 5-20 mg/l.h.

The maximum value of SOUR observed at a temperature of 20°C (Section 6.6) amounted to 25 mg/l.h. As compared to the above, SOUR values of up to 24, 28, 38, 57, 134 mg/l.h were observed at seed concentrations of 100, 250, 500, 750, and 1000 mg/l, respectively. In practice higher values of OUR may be found in sewers due to the activity of slime layer. The SOUR results cited above indicate that seeding sewers with activated sludge (at a concentration of  $\geq 250$  mg/l) would result in a considerable increase in the total oxygen requirements of the wastewater, especially at the point of addition of seed. Considering the fact that an increase in the dissolved oxygen concentration of the wastewater would result in a reduction in the dissolution efficiency of oxygen in sewers (Section 2.4), the results of the current study suggest that the concentration of seed to be added to a given sewerage system may be governed by the ability to dissolve the required amount of oxygen.



## **CHAPTER EIGHT**

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### **CASE STUDY-THE GREATER AMMAN SIPHON**

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#### **8.1. INTRODUCTION**

This chapter describes the results of experimental work carried out at the Central Laboratories of the Water Authority of Jordan, Amman, to study the feasibility of using the 39 km long Greater Amman Siphon (GAS) as an aerobic treatment facility. Batch tests were undertaken, on raw wastewater collected at the inlet of the GAS (Appendix A-1), to simulate the removal of organic matter under aerobic conditions with and without the addition of activated sludge at the inlet of the GAS. A description of the GAS and the experimental methodology adopted for this study are given in Chapter 4 (Section 4.3.2).

#### **8.2. CHARACTERISTICS OF THE WASTEWATER AND SEED SLUDGE**

The removal of soluble organic matter from the GAS wastewater was studied using an initial added seed concentration of 0 (no seeding), 100, 250 and 500 mg VSS/l. A total of four individual grab samples of the wastewaters were subjected to batch tests at each of the above seed concentrations. In the first stage, two batch reactors, the first containing 10 litres of raw wastewater and the second containing 10 litres of raw wastewater seeded with 100 mg VSS/l of activated sludge were run in parallel. In the second stage, two reactors, the first containing raw wastewater seeded with 250 mg VSS/l of activated sludge and the second containing raw wastewater seeded with 500 mg VSS/l of sludge were operated in parallel. The average characteristics of the wastewater studied at different seed concentrations are shown in Table 8.1, while Table 8.2 summarises the average characteristics of the seed sludge. As a result of the addition of activated sludge,

which had a very low soluble COD (SCOD), the SCOD and soluble BOD<sub>5</sub> (SBOD<sub>5</sub>) in the batch reactor at the start of the test were lower than the corresponding SCODs or SBOD<sub>5</sub> values of the raw wastewater. The initial SCODs and SBOD<sub>5</sub> concentrations in the reactor corresponding to each seed concentration are also shown in Table 8.1. The total COD of the influent wastewater averaged 1428 mg/l (SD=328) at initial seed concentrations of 0 and 100, and 1524 (SD=195) mg/l at initial seed concentrations of 250 and 500 mg/l.

**Table 8.1-Average characteristics of GAS wastewater**

Seed concentration	Wastewater characteristics			Initial concentration		n
	SS	SCOD	SBOD <sub>5</sub>	SCOD	SBOD <sub>5</sub>	
0	850±142	395±27	205±26	395±27	205±26	4
100	850±142	395±27	205±26	386±26	200±26	4
250	855±164	360±20	192±24	341±21	185±26	4
500	855±164	360±20	192±24	324±19	172±24	4

Notes: (i) concentration in batch reactor at the start of test  
(ii) all parameters except “n” are expressed in mg/l

**Table 8.2-Average characteristics of seed sludge**

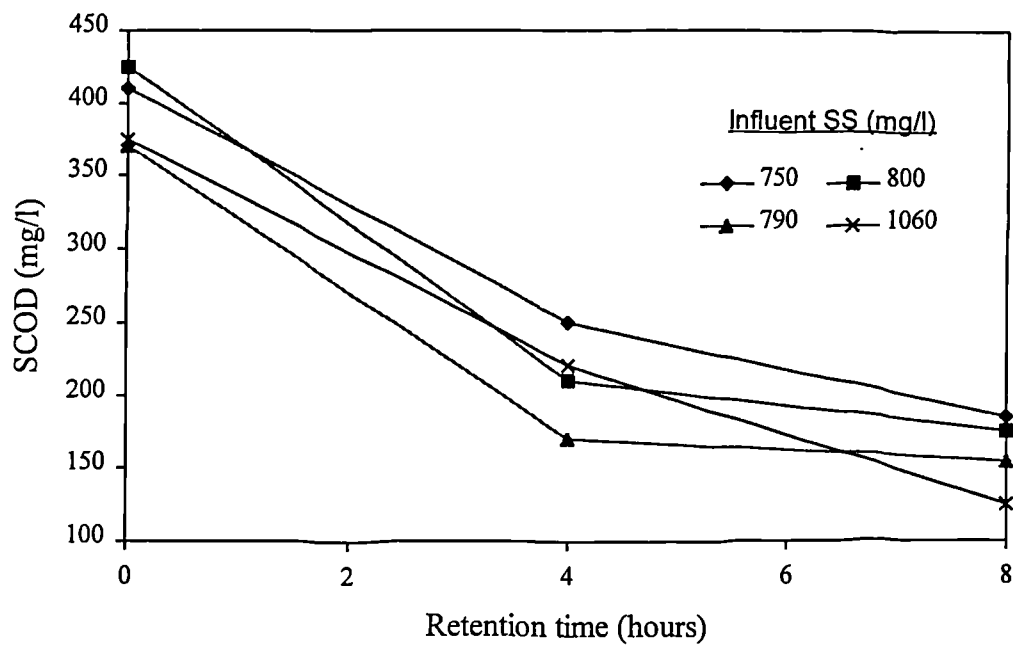
Seed concentration (mg VSS/l)	Seed sludge characteristics (mg/l)			n
	VSS	SCOD	SBOD <sub>5</sub>	
100	3820±184	41±9	8±2	4
250	4645±574	40±9	9±2	4
500	4645±574	40±9	9±2	4

The temperature of the GAS wastewater or the mixed liquor at the start of the experiments ranged from 25-26.5°C at seed concentrations of 0 and 100 mg/l, and 24-30°C at seed concentrations of 250 and 500 mg/l. The temperature of the wastewater in the batch reactors was found to decrease during the test period, depending upon the ambient conditions. The final temperature (after 8 hours) in the batch reactor varied from 20-23°C at seed concentrations of 0 and 100 mg/l, and 20-28°C at seed concentration of 250 and 500 mg/l. The average daily temperatures at each seed concentration however, amounted to approximately 25°C.

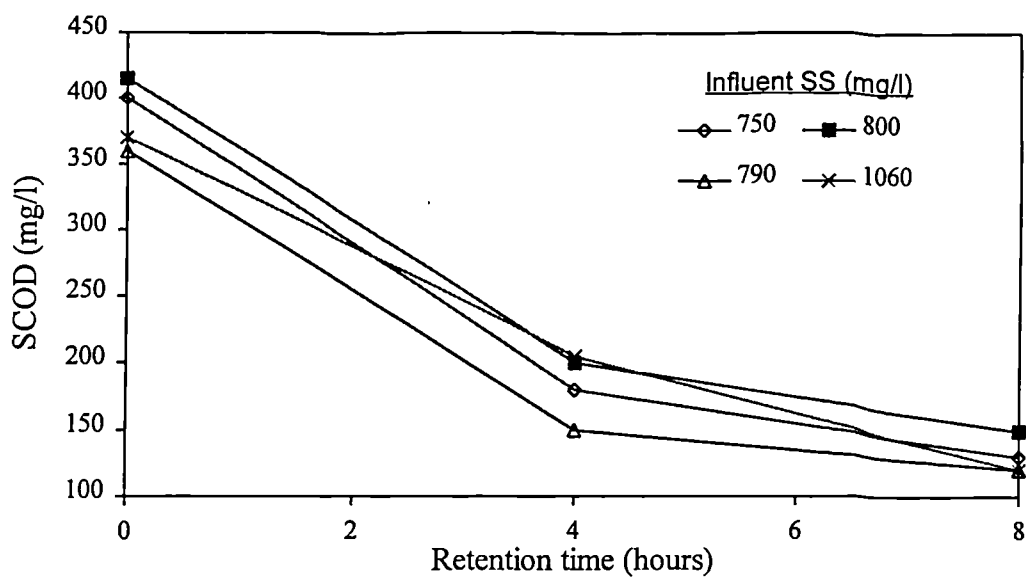
### 8.3. REMOVAL OF SOLUBLE ORGANIC MATTER

Figures 8.1-8.4 show the variation in SCOD over time from the wastewater/mixed liquor at a seed concentration of 0-500 mg/l. The average SCOD and SBOD<sub>5</sub> removal efficiencies at different seed concentrations are summarised in Tables 8.3 and 8.4, respectively.

The average effluent SCODs, after a retention period of 4 hours, amounted to 213 (SD=33), 184 (SD=25), 141 (SD=27), and 110 (SD=16) mg/l. The corresponding values over a retention period of 8 hours were found to be 160 (SD=26), 130 (SD=14), 136 (SD=19), and 114 (SD=16) mg/l, respectively. The corresponding effluent SBOD<sub>5</sub> concentrations, after a retention period of 8 hours, averaged 45 (SD=3), 36 (SD=4), 23 (SD=8), and 12 (SD=10) mg/l, respectively.



**Figure 8.1-Variation in SCOD of GAS wastewater.**



**Figure 8.2-Variation in SCOD of GAS wastewater at an initial added seed concentration of 100 mg/l (Influent SS indicate the SS of the raw wastewater prior to the addition of activated sludge).**

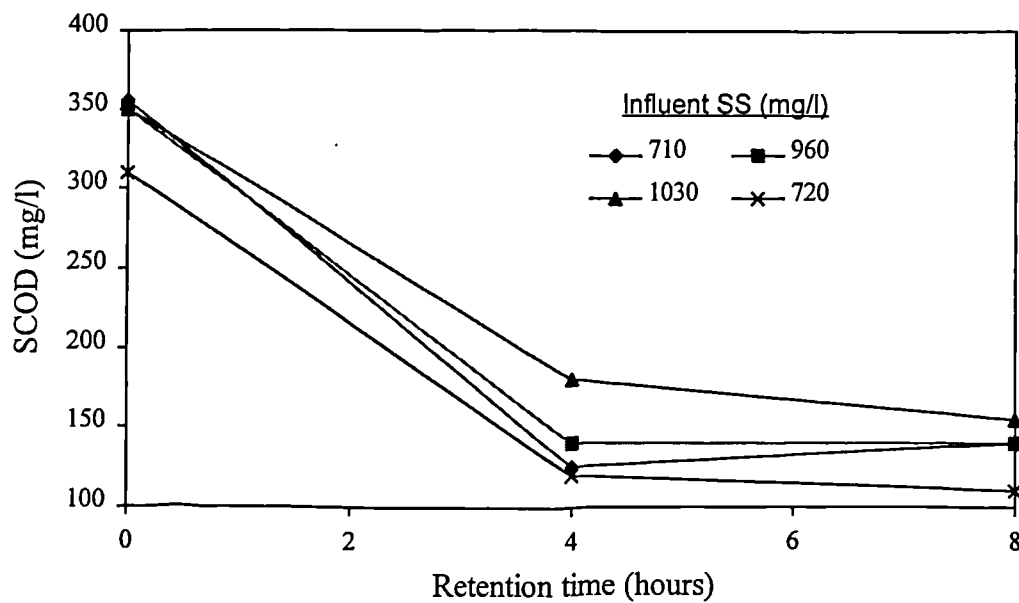


Figure 8.3-Variation in SCOD of GAS wastewater at an initial added seed concentration of 250 mg/l.

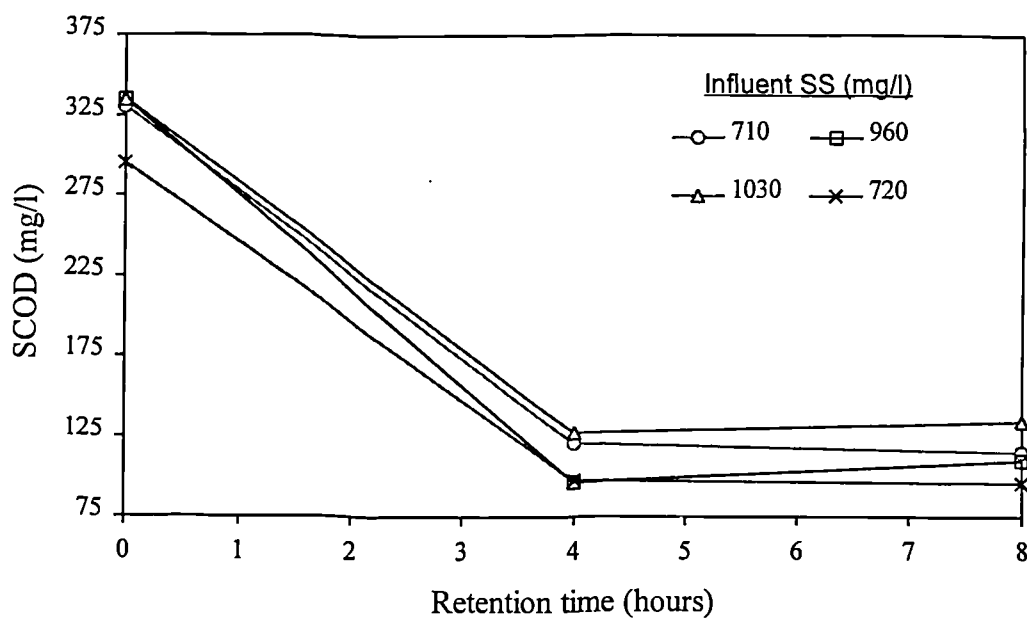


Figure 8.4-Variation in SCOD of GAS wastewater at an initial added seed concentration of 500 mg/l.

**Table 8.3-Average SCOD removal efficiencies**

Seed concentration (mg/l)	Retention time (hours)	SCOD removal (mg/l)	SCOD removal (%)
0	4	183±30	46±7
	8	235±18	60±5
100	4	203±25	52±6
	8	256±14	66±2
250	4	200±26	59±7
	8	205±9	60±4
500	4	214±18	66±4
	8	210±13	65±4

Note : average influent SCOD and SS concentrations are shown in Table 8.1

**Table 8.4-Average soluble BOD<sub>5</sub> removal efficiencies after a retention period of 8 hours**

Seed concentration (mg/l)	SBOD <sub>5</sub> removal (mg/l)	SBOD <sub>5</sub> removal (%)
0	160±25	78±3
100	164±21	82±0
250	161±24	87±4
500	160±24	93±1

Note : average influent SBOD<sub>5</sub> and SS concentrations are shown in Table 8.1

Considering the fact that the use of ATU to arrest nitrification during the BOD<sub>5</sub> test is not practised in Jordan, the effluent BOD<sub>5</sub> concentrations for seeded experiments could be high as they may include the nitrogenous oxygen demand as well.

### 8.3.1. Effect of wastewater strength on SCOD removal

For unseeded experiments (Figure 8.1), the SCOD removal over 8 hours for individual wastewaters, in general, was found to increase with an increase in  $SCOD_0$  or  $SS_0$  of the wastewater. At approximately the same  $SCOD_0$  concentration, i.e. 370 mg/l, an increase in  $SS_0$  of the wastewater, from 790 to 1060 mg/l (34%), increased the SCOD removal over 8 hours from 215 to 250 mg/l (16%). For a similar  $SS_0$  concentration, i.e. 800 mg/l, an increase in initial SCOD from 370 to 410 mg/l (11%) increased the SCOD removal over 8 hours by 35 mg/l (16%). Within the range of values observed during the period of current study, the SCOD removal over 8 hours for seeded experiments did not appear to show any obvious relation with the  $SS_0$  of the wastewater prior to the addition of activated sludge or the initial SCOD of the wastewater.

### 8.3.2. Kinetics of SCOD removal

To find the degradation rate of SCOD in the GAS wastewater at various seed concentrations, first- and second-order reaction rate models were fitted to all the individual curves shown in Figures 8.1-8.4. At seed concentrations of 0 and 100 mg/l, second-order fits yielded higher correlation coefficients than the first-order fits, indicating that SCOD degradation with respect to time would follow second-order kinetics at these seed concentrations.

At seed concentration of 250 mg/l, two wastewaters studied exhibited second order kinetics with respect to SCOD removal over time, while the SCOD removal for the remaining two wastewaters did not show any removal after a retention period of 4 hours (Figure 8.3). A trend similar to the above was also shown by three wastewaters studied at

a seed concentration of 500 mg/l (Figure 8.4). The variation over time in the GAS wastewater at seed concentrations of 250 and 500 mg/l displayed a behaviour similar to that exhibited by wastewater samples collected from Sacriston WWTP at seed concentrations of 750 and 1000 mg/l (Section 7.3; Figures 7.4 and 7.5). Although no SCOD data for the GAS wastewaters were collected between a retention period of 0 and 4 hours, based on the results presented in Section 7.3, it is suggested that the SCOD removal at seed concentrations of 250 and 500 mg/l in the GAS followed second order kinetics with respect to time, during a retention period of  $\leq 4$  hours.

Multiple regression analysis was used to formulate a relationship for describing SCOD removal in the GAS wastewater over time at various seed concentrations. The best-fit empirical equation for SCOD for unseeded wastewater (Figure 8.1) is as follows:

$$\text{SCOD}_t / \text{SCOD}_0 = \log_e (2.7 / (1+t)^{0.27}) \quad (R^2 = 0.97) \quad (8.1)$$

Where:  $\text{SCOD}_t$  = SCOD in the batch reactor at any time  $t$  (1-8 ) hours from the start of the test, and

$\text{SCOD}_0$  = SCOD in the batch reactor at the start of the test.

Equation (8.1) shows a SCOD removal trend similar to that represented by the best-fit equation obtained for Sacriston wastewater at an added seed concentration of 100 mg/l (Equation 7.1). This may reflect the effect of the high strength of the GAS wastewater and the high temperature on SCOD removal pattern.



The following best-fit relationships were obtained for the description of SCOD variation over time at seed concentrations of 100-500 mg/l:

Seed concentration = 100 mg/l:

$$\text{SCOD}_t / \text{SCOD}_0 = \log_e (2.7/(1+t)^{0.31}) \quad (R^2 = 0.98) \quad (8.2)$$

Seed concentration = 250 mg/l:

$$\text{SCOD}_t / \text{SCOD}_0 = (0.3+0.70/(1+t)) \quad (R^2 = 0.98) \quad (8.3)$$

Seed concentration = 500 mg/l:

$$\text{SCOD}_t / \text{SCOD}_0 = (0.23+0.77/(1+t)) \quad (R^2 = 0.98) \quad (8.4)$$

A comparison of the best-fit equation at a seed concentration of 100-500 mg/l for Sacriston wastewater (Equation 7.2), with Equations 8.2-8.4 indicated that at similar seed concentrations, the SCOD removal over time for Sacriston and the GAS wastewater generally showed a similar trend. Consequently, it can be concluded that Equation 7.2 is a reasonable approximation of the SCOD removal over time in a seeded wastewater.

### 8.3.3. Effect of seeding on SCOD removal

Figure 8.5 illustrates the effect of seed concentration on SCOD removal at retention periods of 4 and 8 hours. Figure 8.6 shows the data shown in Figure 8.5 but replotted to illustrate the effect of retention time on SCOD removal efficiency at various seed concentrations.

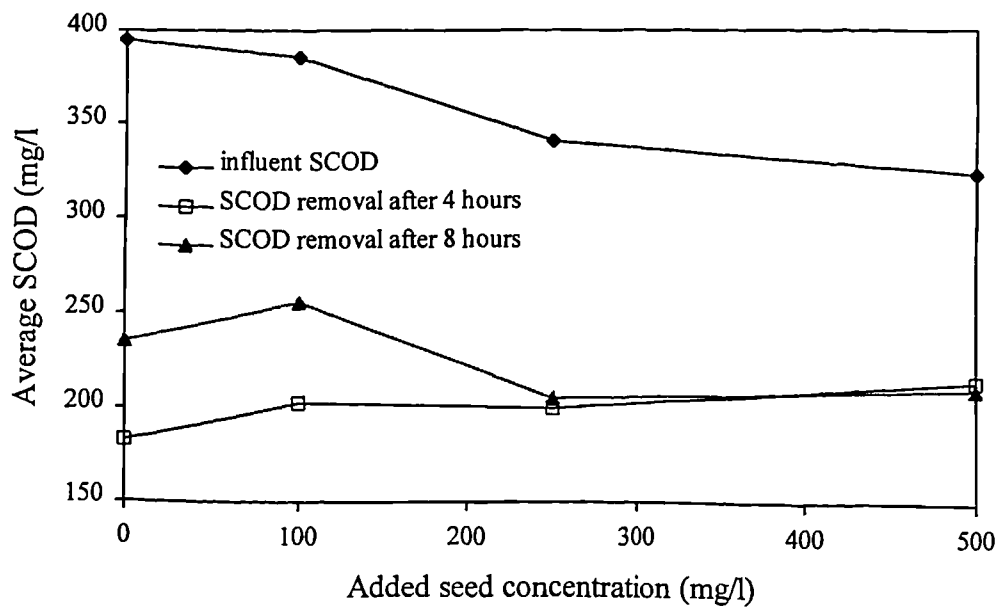


Figure 8.5-Effect of seed concentration on SCOD removal.

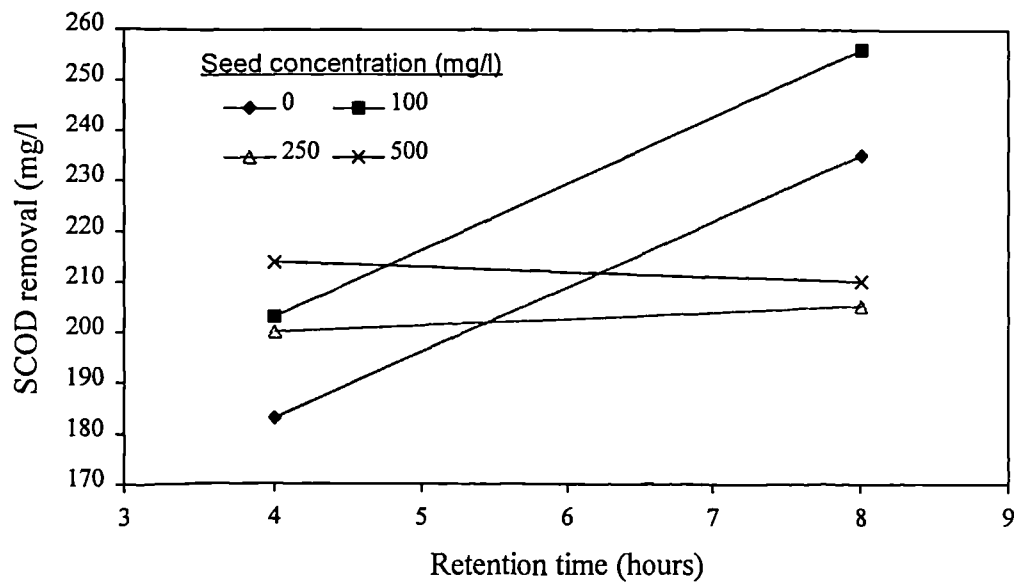


Figure 8.6-Effect of retention time on SCOD removal.

The average improvement in SCOD removal as a result of the addition of 100 mg/l seed, as compared with unseeded wastewaters, averaged 11 and 10% over retention periods of 4 and 8 hours, respectively (Figure 8.5). The average increase in SCOD removal when the seed concentration was increased from 250 to 500 mg/l amounted to 7 and 2.4%, over retention periods of 4 and 8 hours, respectively.

From Figure 8.6, it can be seen that at seed concentrations of 0 and 100 mg/l, the SCOD removal appears to increase linearly with an increase in the retention time from 4 to 8 hours. In contrast, the SCOD removal at seed concentrations of 250 mg/l did not show any noticeable removal after 4 hours, while, at a seed concentration of 500 mg/l, it showed a decrease when the retention period was increased from 4 to 8 hours, due to an increase in the SCOD in the reactor due possibly to generation of soluble microbial products (Section 7.8.1).

It must be noted that a direct comparison of SCOD removal efficiencies for all the seed concentrations investigated can not be made from the results of this study because all the experiments were not run in parallel (Section 8.2), and also the wastewater strength for the seed concentrations of 250 and 500 mg VSS/l was significantly lower than that of 0 and 100 mgVSS/l (Table 8.1). Multiple regression analysis was, therefore, used to evaluate the effect of seed concentration on SCOD removal while taking into account the effect of variation in initial SCOD, and SS concentrations of the individual wastewaters prior to the addition of activated sludge (Appendix E-1). The results of the multiple regression analysis are as follows:

$$\text{RSCOD} = -44.3 + 0.464 \text{SCOD}_0 - 0.0342 \text{SS}_0 + 16.7 t + 25.1 b - 3.2 bt \quad (R^2 = 0.6) \quad (8.5)$$

Where : RSCOD = SCOD removal (mg/l) after a specified retention time  $t$  (hours),

$b = (1 + 0.01 \times \text{seed concentration in mg VSS/l})$ ,

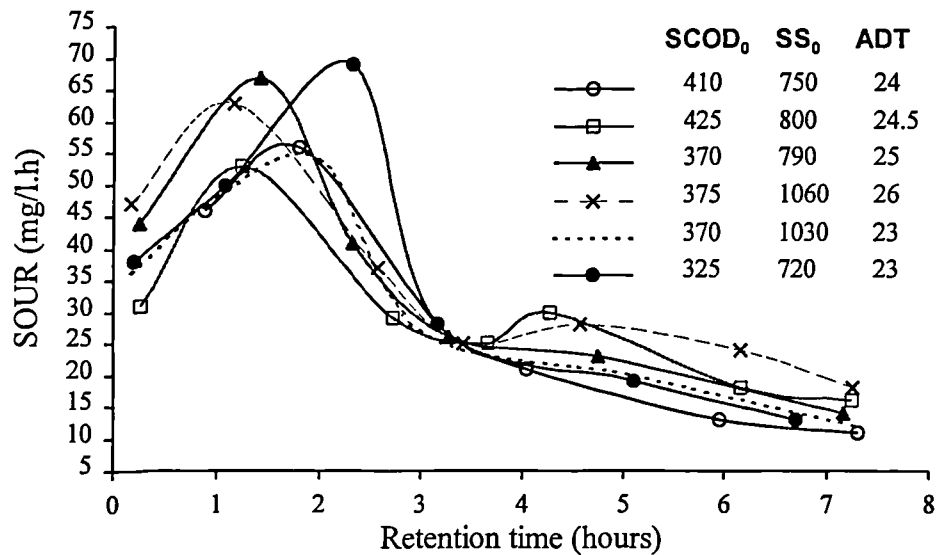
$\text{SCOD}_0$  = SCOD concentration in the reactor at the start of the experiment, and

$\text{SS}_0$  = SS concentration in the wastewater prior to the addition of seed.

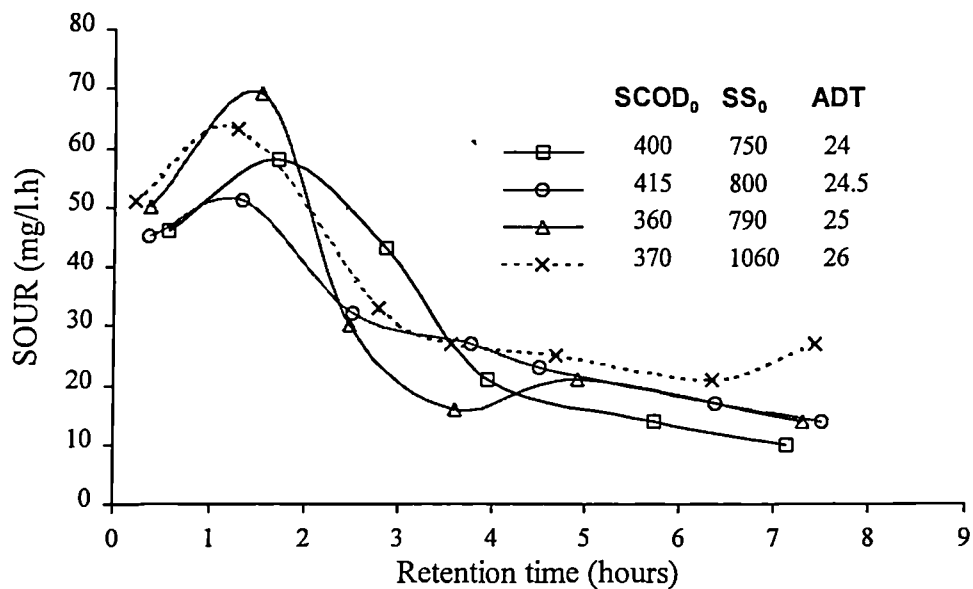
The summary statistics for Equations 8.1 is shown in Appendix E-2. Inspection of summary statistics revealed that at a confidence interval of 5%, the SCOD removal in the GAS wastewater was significantly affected by  $\text{SCOD}_0$ , retention time and seed concentration, and interaction between the seed concentration and the retention period. The effect of  $\text{SS}_0$  however, was not found to be significant.

#### 8.4. OXYGEN UPTAKE RATE OF SUSPENDED BIOMASS (SOUR)

The SOUR profiles of the of wastewater at various seed concentrations, over a retention period of 8 hours, are shown in Figures 8.7-8.10. Table 8.5 shows, for different seed concentrations, the ranges of SOUR observed just after the start and before the end of batch experiments as well as the peak values.



**Figure 8.7-SOUR profile of GAS wastewater** (ADT=average daily temperature °C; SCOD<sub>0</sub> and SS<sub>0</sub> are expressed in mg/l).



**Figure 8.8-SOUR profile of GAS wastewater at an initial added seed concentration of 100 mg/l.**

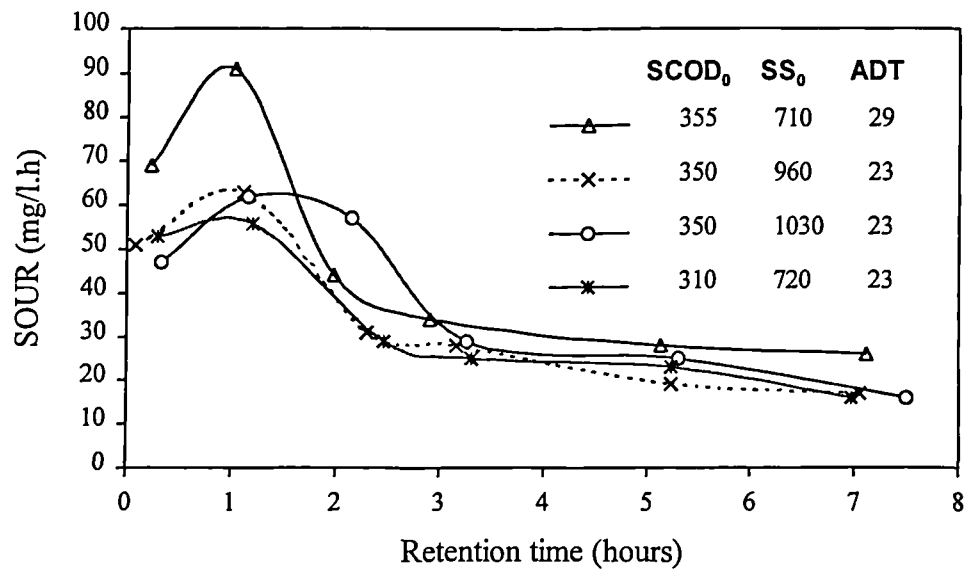


Figure 8.9-SOUR profile of GAS wastewater at an initial added seed concentration of 250 mg/l.

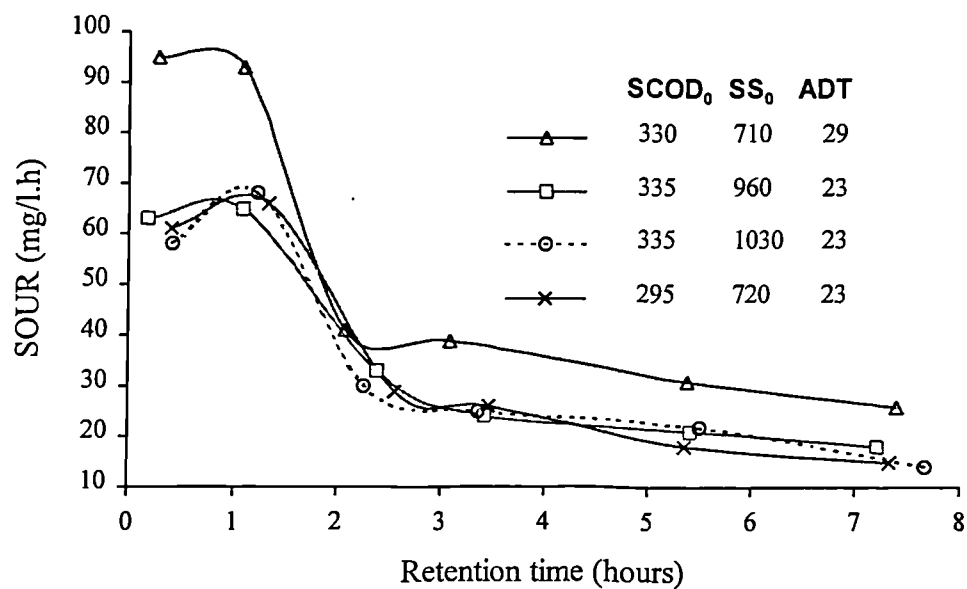


Figure 8.10-SOUR profile of GAS wastewater at an initial added seed concentration of 500 mg/l.

**Table 8.5-SOUR of suspended biomass**

Seed concentration (mg VSS/l)	SOUR range (mg/l.h)		
	Initial <sup>a</sup>	Maximum	Final <sup>b</sup>
0	31-38	53-67	11-18
100	45-51	51-69	10-27
250	47-69	56-91	20-28
500	58-95	65-93	20-28

Notes: <sup>a</sup> during the first hour from the start of oxygen addition

<sup>b</sup> between seven and eight hours from the start of oxygen addition

The total oxygen demand of a given wastewater over 8 hours was determined by computing the area under the SOUR curve for that particular wastewater. Since no SOUR data were collected at 0 and 8 hours, the SOUR curves were therefore extrapolated to include the area between these retention periods. The average SOUR values were determined by dividing the total oxygen demand by the total retention period (8 hours). Table 8.6 summarises the average oxygen demand and SOUR values of the wastewater at different seed concentrations.

**Table 8.6-Average oxygen demand and SOUR values**

Seed concentration (mg VSS/l)	0	100	250	500
Total oxygen demand (mg/l)	240.3±17.9	244±20.9	268.2±34.3	279.6±47.8
Oxygen uptake rate (mg/l.h)	30±2.2	30.5±2.6	33.5±4.3	35±6

From the results shown in Figures 8.7-8.10, it can be seen that the SOUR at seed concentrations of 0-500 mg/l, was, in general, low just after the start of the oxygen addition, it increased to a maximum value afterwards, and then showed a gradual

decrease. The peak values of SOUR appeared to have occurred when the retention period was between 1-2.5 hours for seed concentrations of 0 and 100 mg/l, and < 1.5 hours at seed concentrations of 250 and 500 mg/l.

From the results presented in Table 8.5, it can be seen that an increase in seed concentration tends to increase the initial SOUR of the wastewater. It was, however, not possible to isolate the effect of seed concentration on overall SOUR (average over 8 hours) due to the variation in SCODs and temperature of the wastewater.

## **8.5. DISCUSSION**

### **8.5.1. Removal of soluble organic matter**

The results of the current study demonstrate that by maintaining aerobic conditions in the GAS, SCOD and SBOD<sub>5</sub> removals of 60 and 78% respectively, could be achieved at an average temperature of 25°C and over a retention period of 8 hours without seeding the GAS. There was also evidence to suggest that the corresponding substrate removal efficiencies could improve to 66 and 82% respectively if the GAS wastewater were to be seeded with activated sludge at a concentration as low as 100 mg VSS/l. From the results shown in Table 8.3 it also appears that the time required to achieve SCOD reductions equivalent to those obtained over 8 hours with unseeded wastewater may be reduced to ≤ 4 hours when the seed concentrations is increased to 250-500 mg/l. Moreover, average soluble BOD<sub>5</sub> removal efficiencies of 87 and 93% were achieved at seed concentration of 250 and 500 mg/l respectively (Table 8.4), which are comparable to those found in a typical activated sludge plant.



SCOD removal efficiencies over 4 hours for the unseeded wastewater collected from Durham WWTP (Chapter 6) averaged 134 mg/l and 144 mg/l at 20 and 30°C, respectively. The corresponding removal efficiencies over 8 hours averaged 186 and 191 mg/l, respectively. When compared to the Durham wastewater (having a high SCOD<sub>0</sub> and high SS<sub>0</sub>), the SCOD removal efficiencies for the unseeded GAS wastewater averaged 183 and 235 mg/l over a retention period of 4 and 8 hours, respectively, at an average temperature of 25°C. These values are 37 and 27% higher than the corresponding values over 4 hours for the Durham wastewater at 20° and 30°C, respectively. Similarly, the SCOD removals in the GAS over 8 hours were 26 and 23% higher than the corresponding values found for Durham wastewaters at 20 and 30°C, respectively. Also, the SCOD removal over time for unseeded GAS wastewater, when compared with unseeded wastewater studied at Newcastle (Chapter 6), was found to follow second-order kinetics. The high SCOD removal efficiencies for the GAS wastewater and second order SCOD removal kinetics may be attributed to the high strength of the wastewater, especially the high SS (average 850 mg/l) concentration. These results demonstrate that the strength of the wastewater would be a major parameter affecting in-sewer treatment.

The results of the Amman study showed that the effect on SCOD removal of seeding the wastewater with activated sludge in the range of 100-250 mg/l was marked only when the retention period was 4 hours, possibly due to the fact that, for seeded experiments, most of the SCOD was removed over this period. A diminished effect of seeding with an increase in seed concentration and retention time is consistent with the findings of previous investigations which involved wastewaters taken from Sacriston WWTP, in Newcastle (Chapter 7). A number of possible explanations for the diminished effect of seed at higher retention periods have been outlined in Chapter 7 (Section 7.8.1).

Considering the fact that under limiting substrate conditions, the amount of SCOD removal in a batch reactor is proportional to the amount of SCOD remaining in the reactor, one of the reasons for low SCOD removal efficiencies at seed concentrations of 250 and 500 mg/l noted in the current study may be the low initial SCOD concentrations at these seed concentrations.

### 8.5.2. Oxygen uptake rate

When compared with the SOUR profiles for unseeded wastewaters studied in Newcastle (Section 6.6), the SOUR profiles for unseeded GAS wastewater showed a regular pattern with respect to time (Figure 8.7). This may be due to the fact that the samples of the GAS wastewater used for batch tests were relatively young as they were collected at the inlet to the sewer rather than the inlet to the wastewater treatment works, as was the case for Newcastle study. The SOUR results observed in the current study support the view expressed by other researchers (Pomeroy and Parkhurst, 1972; Boon and Lister, 1975) that the SOUR of a fresh wastewater would exhibit a bell-shaped profile (Figure 8.7) over time.

The maximum values of SOUR observed for wastewaters studied in Newcastle (Section 6.6) were found to be 25 and 40 mg/l.h at 20° and 30°C respectively. When compared to the above, peak values ranged from 53-67 mg/l.h for the unseeded GAS wastewater, while the SOUR over 8 hours averaged 30 mg/l.h at a temperature of 25°C. At a similar temperature (24-28 °C), Hemmings *et al.* (1983), based on field studies, reported an average wastewater oxygen uptake rate of 14 mg/l.h. Considering the above, a high SOUR value for the GAS wastewater may be attributed to its high strength.

The average daily temperature of the wastewater did not vary greatly except for the experiments conducted at seed concentrations of 250 and 500 mg/l. Although the variation in temperature did not appear to show any noticeable effect on SCOD removal over retention periods of 4 and 8 hours, it did significantly affect the SOUR of the wastewater as can be seen from Figures 8.9 and 8.10. At a seed concentration of 250 mg/l, the total oxygen demand of the wastewater over 8 hours increased from 235-273 mg/l to 322 mg/l when the average daily temperature increased by 6°C (from 23°-29°C). Similarly, a 6°C rise in temperature, at a seed concentration of 500 mg/l, increased the total oxygen demand from 235-273 to 322 mg/l. These results demonstrate that the diurnal variation in the temperature of the GAS wastewater could produce marked changes in the oxygen requirements of the wastewater, and thus provision should be made in the design to accommodate such variation.

### **8.5.3. Prospects of using the GAS as a treatment facility**

The data collected during the course of this study, although limited, have produced evidence which demonstrates that maintaining aerobic conditions in the GAS could result in a considerable degree of in-sewer treatment, and may also alleviate the serious odour problems being encountered presently at the inlet of AWSP system. From the data presented herein it also appears that, seeding the GAS (at a concentration of about 250 mg VSS/l) at the inlet could also result in removal efficiencies comparable to those obtained in an aeration tank of a conventional activated sludge plant. In this case, the existing abandoned activated sludge plant at the inlet of the GAS could be re-activated to serve as a feasible source of seeding. A portion of flow could be drawn and treated in the above treatment plant, and the excess sludge then could be added to the inlet of the sewer.

Considering an average SOUR value of 30 mg/l.h (Table 8.6), the total oxygen demand of the wastewater (including the oxygen uptake rate of the slime layer) as calculated from Equation 2.3 (Boon and Lister, 1975), would be approximately 34.5 mg/l.h or 276 mg/l over a retention period of 8 hours at 25°C. At an average daily flow of 115,000 m<sup>3</sup>/day (Section 4.3.2), the total oxygen demand of the wastewater (for the unseeded option) would amount approximately to 32 tons/day.

When compared to the UK where the cost of liquid oxygen delivered at site varies from £37-£80 per ton (BOC, 1994), the cost of liquid oxygen delivered to site in Jordan varies from £200-£500 per ton. The cost of oxygen injection for unseeded option therefore, may exceed £6400-£16000 per day or £2,336,000-£5,840,000 per year. It is however, likely that this cost may be reduced by on-site generation of oxygen. The use of GAS as a seeded treatment facility with a view to enhancing the in-sewer treatment, when compared with the unseeded option, would entail higher capital and running costs including the cost of increased oxygen requirements and the cost of seed generation.

Considering a high oxygen demand of the GAS wastewater, and the fact that the dissolution efficiency of oxygen decreases with an increase in temperature and the target dissolved oxygen (Sections 2.4 and 2.8), it appears that the use of oxygen as a source of DO for the GAS would be favoured in the place of air. Also, it seems that the oxygen injection facilities have to be located at many points along the GAS as it would not be economically feasible to dissolve such a high DO concentration (276 mg/l) at a single point. A high pressure at the bottom of the siphon may however, help minimise the number of oxygen injection points along the rising leg of the siphon.

As mentioned earlier, the cost of oxygen in Jordan is considerably higher than that in the UK or other countries where in-sewer oxygen injection technology is well developed. Also, no in-sewer oxygen injection technology is currently available in Jordan and the use of the GAS as an aerobic treatment facility would therefore, require sophisticated equipment to be imported from abroad, which may not be economically viable.

Considering the high cost of oxygen, the absence of in-sewer oxygen injection technology and the lack of world-wide experience in the field of in-sewer treatment, it appears that the use of GAS as a treatment facility, for the time being, may not be favoured, when compared with the proposed short-term extension of the existing wastewater stabilisation pond system. Subsequent to the signing of the peace agreement between Israel and Jordan however, huge capital investment is being made in various sectors in Jordan by a number of European and American companies. This is likely to solve the problem of oxygen injection technology and bring down the cost of oxygen generation, and hence, may in the long-term, improve the prospects of using the GAS as a treatment facility.

The results of the present study however, have advanced sufficient evidence in support of the use of GAS for in-sewer treatment to warrant further research work, using more precisely designed experiments. A detailed technical and economic evaluation of the GAS as a treatment facility would require further experimental work aimed at investigating, both the seeded and unseeded options, the effect of diurnal and seasonal variation in wastewater strength and temperature on in-sewer treatment, oxygen uptake rate of the wastewater and slime layer, settling characteristics of the effluent, and sludge production.

## CHAPTER 9

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### GENERAL DISCUSSION

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#### 9.1. INTRODUCTION

The results of the current study have demonstrated that sewers may be used as aerobic biological reactors to effect at least partial treatment of wastewater during transit, and could under certain circumstances, replace the aeration units of an activated sludge plant. The main factors affecting in-sewer treatment have been found to be wastewater strength, retention time, temperature, and the seed concentration (in those cases where it is used).

The design of an existing sewerage system or a proposed system for in-sewer treatment however, would require a number of factors to be considered in addition to the parameters referred to above and some of which may be involved in assessing the potential of a sewerage system for in-sewer treatment are discussed in the following sections.

#### 9.2. NETWORK LAYOUT AND CHARACTERISTICS

The type, layout and characteristics of a sewerage system would be a critical element in assessing its feasibility for in-sewer treatment, either technical or economic, in any given situation. There are many scenarios, with consideration being given to its potential for use from partial to complete secondary treatment. The following are a few such scenarios which may be considered:

### 9.2.1. Pressure mains versus gravity sewers

All the field studies on in-sewer treatment conducted so far have involved pressure mains (Section 2.2.2). The major reasons for the selection of pressure mains over gravity sewers could be the well-developed air or oxygen injection technology and the ability to attain high dissolved concentrations under a high pressure. Also, better operational control possibly favours the use of pressure mains while the situation in gravity sewerage systems is more complex because of flow variations and the presence of tributaries. When compared with pressure mains, the changes in flow in gravity sewers result in changes in the effective slime area in contact with the wastewater and hence the degree of treatment that can be achieved. An estimate of the treatment in gravity sewers however, could be made based on minimum, average and maximum flow rates (Newcombe *et al.* 1979) .

In addition to the advantages described above, it has been suggested that the use of a pressure main for in-sewer treatment may result in a higher degree of in-sewer treatment than in gravity sewers. Based on batch studies, undertaken to simulate a step-fed sewer, Green *et al.* (1985) reported that the average soluble COD removal efficiency increased by 7% when the reactor was operated under pressure (2 atmospheres) for 3 hours out of a total retention period of 10 hours. They attributed increased substrate removal to the decreased floc size as a result of pressure, which could cause an increase in substrate availability to the inner parts of the floc, and also provides a larger surface area. Lin and Huang (1993), based on laboratory studies, also claimed that the efficiency of an activated sludge process could be significantly improved by pressurised aeration.

### **9.2.2. Long, single pressure pipe**

A long single pressure pipe could offer a significant potential for partial treatment of a wastewater, even without seeding, and may even offer treatment potential comparable to that in a secondary treatment plants when the temperature is high. Typical examples of such systems include a 40 km long pressure main in Kuwait, and a 40 km long siphon in Amman, Jordan.

An alternative configuration of the long, single pressure pipe would be the use of a specifically constructed pressure pipe which treats the wastewater while it is being transported to an upstream point where there is a potential for its re-use. Such a system, proposed initially by Stoyer (1970), may be particularly applicable to some current situations in the UK. It may be possible to supplement low river flows, which are becoming an increasing concern of the Environmental Agency, by returning a wastewater to an upstream point. Not only would this increase the potential for in-sewer treatment, it would also prevent the 'loss' of water to coastal areas.

### **9.2.3. Loop system**

The sewerage system may be such that the location of the downstream wastewater treatment plant is in reasonably close proximity to the head of the system and a return activated sludge pipe could readily and cheaply be constructed to introduce activated sludge at the start of the system. Such a sewerage system exists in Tel Aviv, Israel, where the treatment work is at a distance of only 8 km from the inlet of a 37 km long sewer (Green *et al.*, 1985). Manandhar and Schroder (1995) have proposed the use of loop



sewerage system as a circulating reactor wherein the wastewater could be recycled to its point of origin to enhance self-purification.

#### **9.2.4. Upstream wastewater treatment plant**

It may be viable for a small upstream treatment plant to be constructed which may draw off a portion of the wastewater flow from the sewer for treatment. The excess activated sludge could then be returned to the sewer as the source of biomass for in-sewer treatment.

#### **9.2.5. Upstream wastewater treatment plant on unconnected sewerage system**

Due to a number of factors including population distribution, topography, historical development etc., it may be possible for a treatment plant to be located close to the head of another sewerage system. The trunk sewer being considered for in-sewer treatment could use the excess sludge as the required biomass, as well as being used for disposal of the sludge.

### **9.3. ROLE OF THE SLIME LAYER IN IN-SEWER TREATMENT**

As already indicated, sewers constitute a dual-phase system with suspended biomass present in the liquid phase and in the slime layer on the wetted surface of the sewer wall. The present study focused on the removal of soluble organic matter in the liquid phase only. The in-situ substrate removal rate, therefore, may be higher than those observed in the current study due to the activity of the micro-organisms present in the slime layer.

The contribution of the slime layer to the removal of organic matter in sewers is significant only for shallow sewers (Pomeroy and Parkhurst, 1972) and is governed by the ratio of the area of wetted surface (A) to the wastewater volume (V), and on the suspended solids concentration in the wastewater (Cao *et al.* 1992). Based on laboratory studies designed to investigate a shallow river system, Cao *et al.* (1992) demonstrated that an increase in A/V resulted in a corresponding increase in the contribution of the biofilm to the removal of organic matter, relative to that of the liquid phase. It was however, noted that at an A/V ratio of  $2.1 \text{ m}^{-1}$  the biofilm played a major role only when the suspended solids concentration was  $\leq 12 \text{ mg/l}$ . The specific activity of suspended biomass with respect to oxygen consumption rate was found to be 50 to 100 per cent of that of the biofilm. On the basis of laboratory studies, Raunkjaer (1993) demonstrated that at an A/V ratio of  $37 \text{ m}^{-1}$ , the removal from the biofilm accounted for 70% of the total organic matter removal from the liquid phase and the biofilm, at  $16^\circ\text{C}$ .

#### 9.4. IN-SEWER TREATMENT FOR DEVELOPING COUNTRIES

Much of the developing world is currently without sewerage or any form of wastewater treatment. Moreover, the problems of wastewater collection, treatment and disposal is generally magnified in developing countries since they are frequently in warm climates and the wastewater strength is often significantly greater than that in the UK.

The potential of in-sewer treatment for developing countries appears to be great considering that, in many cases, completely new sewerage systems are being developed which could be "engineered" to optimise in-sewer treatment. Another aspect which is likely to improve the prospects of in-sewer treatment in developing countries is the high

temperature (greater than 25°C) and the need anyway to maintain higher DO levels for sulphide, odour and corrosion control. Many large, new schemes are also being constructed in areas with little or no natural fall, thereby increasing the retention time and increasing the need for oxygen supplementation.

### **9.5. OPERATIONAL PROBLEMS ASSOCIATED WITH IN-SEWER TREATMENT**

Although the use of sewers as a treatment facility would prove beneficial in certain situations, it could adversely effect nutrient removal at a downstream treatment plant incorporating such a system.

Another adverse effect of in-sewer treatment would be the “decentralisation” of operation and maintenance since much of the aeration and pumping equipment would be located at points remote from the main wastewater treatment plant. This, however, does not detract from the overall attractiveness of the concepts discussed in this dissertation.

### **9.6. ECONOMICS OF IN-SEWER TREATMENT SYSTEM**

Compared with an aeration basin in a conventional secondary treatment system, the land-take of an in-sewer aerobic treatment (ISAT) system, in the case of an existing sewer, would be negligible, while the corresponding running costs would be mainly associated with the cost of oxygen injection. The overall economics of ISAT, however, will be governed by a number of factors including the degree of treatment which can be achieved, the availability of adequate oxygen injection techniques, the cost of sludge recycling, in addition to the factors outlined in Section 9.2, and would be highly site-specific.

In-situ studies have demonstrated that a considerable saving in investment could be achieved by using sewers for partial or full aerobic treatment. Newcombe *et al.* (1979) concluded that by using an 8 km long rising main to upgrade a plant overloaded by 40 % would result in a saving of £188,000 in NPV (net present value) over 20 years, while producing an effluent quality of 50/30 (SS/BOD<sub>5</sub>) in winter and 30/20 in summer. The overall saving was found to be as a result of the prevention of septicity and savings in the cost of conventional extensions.

The use of a 6 km long pressure main followed by a sedimentation tank as a full treatment facility in Australia was found to produce a 50/50 (SS/BOD) effluent (Hutchinson, 1981; Hemmings *et al.*, 1983). It was estimated that, in exchange for an additional ongoing running cost of Aust. \$20,000 per annum, largely associated with oxygen injection, there would be a saving of Aust. \$700,000 in initial capital outlay for conventional secondary treatment facilities to achieve a similar effluent quality. Stoyer (1970) and Green *et al.* (1985) also estimated that by using a sewer followed by a sedimentation tank for full treatment instead of a conventional activated sludge plant, a saving of more than 50 % could be expected in the construction costs.

## CHAPTER 10

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### CONCLUSIONS

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The following main conclusions can be drawn from the results of this simulation study designed to investigate the in-sewer treatment of domestic wastewater:

#### 10.1. AEROBIC VS. ANAEROBIC CONDITIONS IN SEWERS

- A comparison of simulated aerobic and anaerobic gravity transport of raw domestic wastewater demonstrated that aerobic conditions in sewers, when compared with anaerobic conditions, could result in a significantly greater removal of soluble organic matter together with improved settleability of primary solids at the downstream treatment works. This would suggest that aerobic treatment would be the most favoured method of in-sewer biological treatment.

#### 10.2. EFFECT OF MIXING SPEED ON IN-SEWER AEROBIC TREATMENT

- During the simulated aerobic gravity transport of wastewater, the effect of mixing speed (which is in turn related to the wastewater velocity), in the range 50-300 rpm, did not appear to have a significant effect on the removal of soluble COD nor on suspended solids. At higher mixing speeds (>200 rpm) however, there was a statistically significant reduction in suspended solids removal in a bench-scale settling column, due to increased turbulence in the reactor which caused the the flocs to break-up.

### **10.3. EFFECT OF DISSOLVED OXYGEN SOURCE ON IN-SEWER AEROBIC TREATMENT**

- A comparative study designed to investigate the effect of dissolved oxygen source (air or oxygen) on the reduction in soluble COD from raw wastewater during simulated aerobic gravity transport, and on suspended solids removal at the downstream treatment works, showed that the effect of source would not be statistically significant.

### **10.4. ADDITION OF OXYGEN TO RAW WASTEWATER DURING SIMULATED GRAVITY TRANSPORT AT 20°C**

- At a given temperature, the removal of soluble organic matter from wastewater was found to be strongly influenced by the suspended solids and soluble organic matter present in the wastewater as well as by the hydraulic retention time. Over a retention period of 8 hours at 20°C, the soluble COD removal efficiencies averaged 48, 40, and 61% for wastewaters having average influent soluble COD concentrations of 176, 329 and 306 mg/l, and influent suspended solids concentrations of 353, 346 and 620 mg/l, respectively.
- At 20°C, the soluble COD removal from the wastewater was found to be significantly influenced by the influent suspended solids (but not the influent soluble COD) over short retention periods i.e. 1-3 hours. For higher retention periods ( $\geq 4$  hours), the soluble COD removal was significantly affected by both the influent suspended solids and the influent soluble COD of the wastewater.

- The soluble COD removal from the wastewater, within a temperature range of 10-30°C, mainly followed first-order kinetics with respect to time. Based on the experimental results, an empirical relationship for calculating the soluble COD removal efficiency for a given strength of wastewater was formulated.
- The temperature coefficient for soluble COD removal rate from wastewater averaged 1.088 and 1.056 within the temperature ranges of 10°-20°C and 15°-30°C, respectively.
- Over a retention period of 8 hours, the total COD removal from the raw wastewater (unsettled) varied from 6-23% at 20°C, and did not appear to show any relation to the influent total COD or influent suspended solids. These results suggest that, when compared with soluble COD removal, the total COD removal from the raw wastewater would be much lower.
- Over a retention period of 6 hours at 20°C, the changes in suspended solids of the wastewater did not show any clear pattern, ranging from -28 to +19%.

#### **10.5. ADDITION OF OXYGEN TO RAW WASTEWATER ADDED WITH ACTIVATED SLUDGE, DURING SIMULATED GRAVITY TRANSPORT AT 20°C**

- Addition of activated sludge to the wastewater in order to increase the suspended biomass concentration, at initial levels of as low as 100 mg VSS/l, resulted in a

significant increase in soluble organic matter removal, and hence a significant reduction in the retention time required to achieve a given degree of treatment.

- The soluble COD removal was found to increase almost linearly as the activated sludge concentration added to the wastewater increased in the range of 100-1000 mg/l. The effect of increasing the seed concentration on soluble COD removal however, appeared to diminish with the increase in retention time.
- The soluble COD and suspended solids concentrations present in the wastewater prior to the addition of activated sludge, the concentration of activated sludge added to the wastewater and the hydraulic retention time were all found to have significant effect on soluble COD removal from the wastewater. In addition, the interaction between activated sludge concentration and the hydraulic retention time was also found to have a significant effect on soluble COD removal.
- Within a range of activated sludge concentrations of between 100-1000 mg/l, the soluble COD removal from the wastewater appeared to follow second order kinetics with respect to time. Based on the experimental data collected in the current study, empirical equations were developed for the description of variation in soluble COD over time.
- At activated sludge concentration of 100-1000 mg/l, the effluent from the batch reactor after a retention period of 6 hours, showed satisfactory settling characteristics i.e. having average SVI values of less than 150 ml/g. The SVI of the mixed liquor,



however appeared to increase with an increase in seed concentration. The settleability results also suggested that, despite low SVI values, the solids removal at activated sludge concentrations of 100-250 mg/l may be poor due to lack of adequate bioflocculation, although the nature of the SVI test may be a significant factor in this case.

- The oxygen uptake rate of the seeded wastewater did not show any specific trend over time at activated sludge concentrations of 100 and 250 mg/l while at higher concentrations it displayed a pattern similar to those observed in the aeration tank of a plug flow activated sludge plant. The oxygen uptake rate, especially just after the addition of the activated sludge, increased significantly with an increase in activated sludge concentration. At activated sludge concentrations of  $\geq 250$  mg/l, the oxygen uptake rate was found mainly to be influenced by the hydraulic retention time. At a sludge concentration of 1000 mg/l however, the effect of influent soluble COD was also found to be significant.

#### **10.6. CASE STUDY-THE GREATER AMMAN SIPHON**

- It has been demonstrated that, by maintaining aerobic conditions in the Greater Amman Siphon, average soluble COD and soluble BOD<sub>5</sub> removal efficiencies of 60% and 78% respectively, could be achieved over a retention period of 8 hours at 25°C.
- The oxygen uptake rate of the unseeded Amman wastewater averaged 30 mg/l.h. The total oxygen demand of the wastewater was estimated to be approximately 32 tons/day at an average flow rate of 115,000 m<sup>3</sup>/day.

- There was evidence to suggest that by seeding the Amman wastewater, by the addition of activated sludge at the inlet of the Greater Amman Siphon, at a level of 250-500 mg VSS/l, the soluble organic matter removal comparable to that found in the aeration tank of a typical activated sludge plant, could be achieved.
- The overall results of the case study indicate that the Greater Amman Siphon has a great potential for being used as a long term solution to a reduction in the load on the Al-Samra wastewater stabilisation ponds system. Further studies however would be required before a detailed technical and economic evaluation of the siphon as a treatment facility could be undertaken, recommendations have been made in this regard.

## CHAPTER 11

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### RECOMMENDATIONS FOR FURTHER WORK

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Based on the results of the current study, the following recommendations are made for future research on in-sewer treatment:

1. Pilot-scale studies using fresh raw domestic wastewater should be undertaken to investigate the treatment potential of sewers with and without seeding. For this purpose, a pipe could be laid parallel to an existing sewer, and fed with fresh wastewater using a pump. The length of the pipe could be minimised by laying the pipe in the form of a loop in which wastewater may be circulated for the desired retention period. The main parameters of interest would include:
  - (i) the effect of diurnal and seasonal variation in wastewater strength and temperature on in-sewer treatment and on the oxygen uptake rate of the wastewater and the slime layer,
  - (ii) settling characteristics of the effluent after discharge from the sewer,
  - (iii) sludge production rates and biological sludge viability,
  - (iv) effect of changes in velocity and flow variations,
  - (v) effect of the type of the sewer i.e. pressure vs. gravity, and linear vs. branched,
  - (vi) bacterial die off rates,
  - (vii) use of chemicals instead of air or oxygen,
  - (viii) relation between the oxygen uptake rate and wastewater strength,
  - (ix) relative organic matter removal rates in the liquid phase as well as by the slime layer,

- (x) continuous vs. intermittent air or oxygen injection, and
  - (xi) the effect of in-sewer treatment on the efficiency of nutrient removal unit processes.
2. A comparative study should be undertaken to investigate the performance of various in-sewer aeration or oxygenation devices for in-sewer treatment.
  3. The treatment potential of sewers should be considered at the design stage and attempts should be made to design and lay the new sewerage systems such as to optimise the in-sewer treatment.
  4. A survey of existing sewer systems should also be undertaken to identify those with potential for in-sewer treatment. The economics of in-sewer treatment in comparison with the costs of extension of existing overloaded wastewater treatment plants should also be assessed.
  5. A mathematical model should be developed for the design and operation of in-sewer treatment system.

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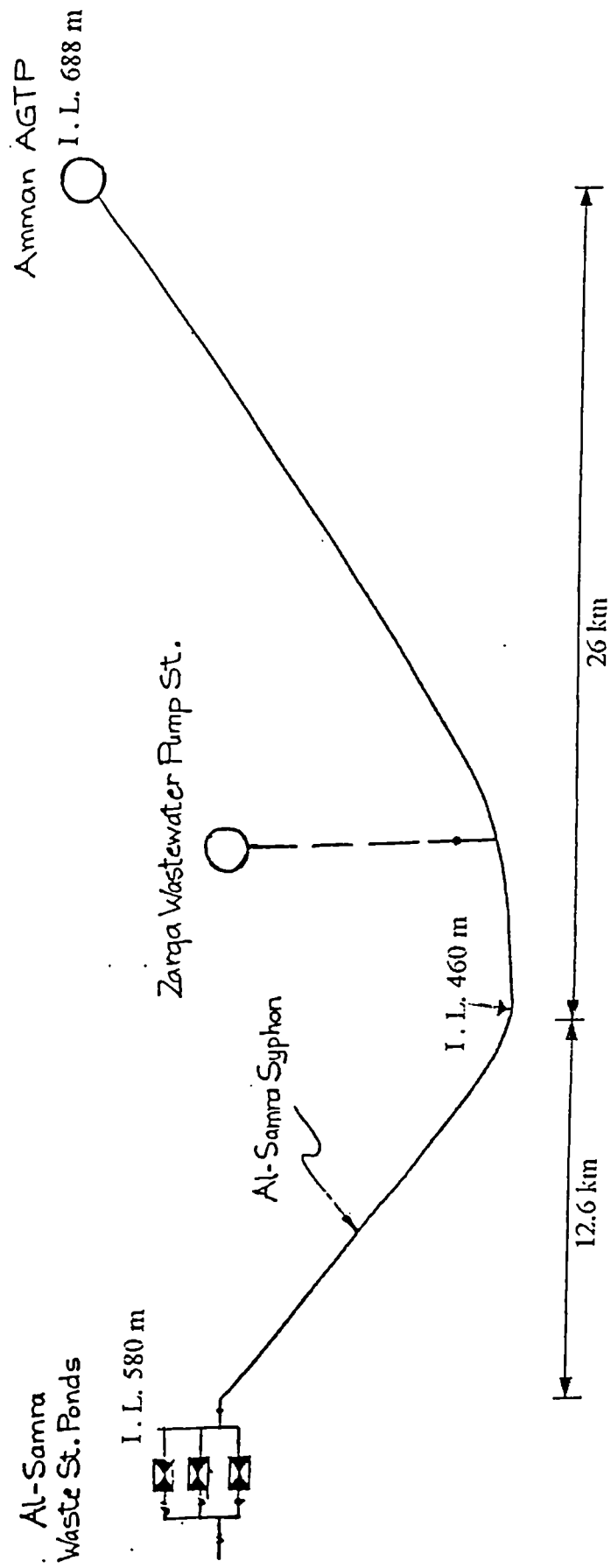
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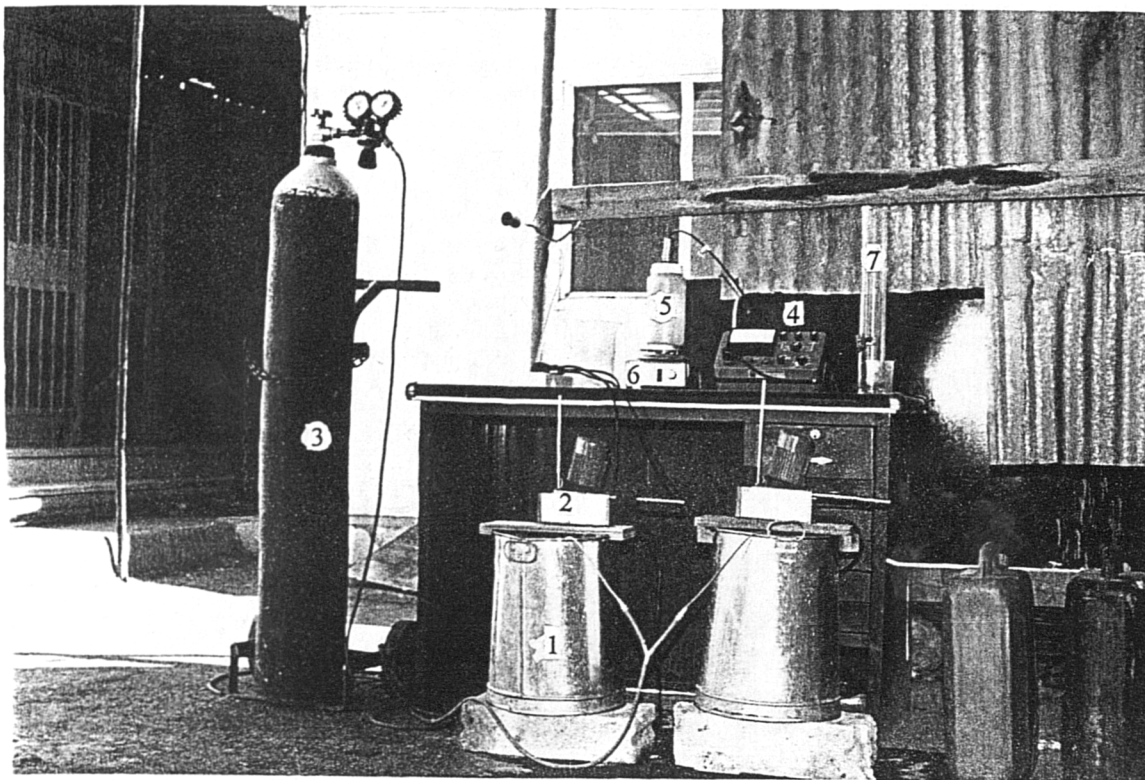
# APPENDIX A-1

## The Greater Amman Siphon



## APPENDIX A-2

### Experimental set-up for Phase-II studies



1. Batch reactor; 2. Impeller; 3. Oxygen cylinder; 4. Do meter; 5. SOUR reactor; 6. Magnetic mixer; 7. Settling column.



## APPENDIX B-2

## RESULTS OF ANOVA TESTS FOR THE EFFECT OF AIR AND OXYGEN

MTB &gt; Oneway 'RSCOD(%)' 'source'.

SOURCE	DF	SS	MS
source	1	18.0	18.0
ERROR	6	166.0	27.7
TOTAL	7	184.0	

F	p
0.65	0.451

INDIVIDUAL 95% CI'S FOR MEAN  
BASED ON POOLED STDEV

LEVEL	N	MEAN	STDEV
1	4	36.000	5.477
2	4	39.000	5.033

POOLED STDEV = 5.260

30.0	35.0	40.0	45.0
INDIVIDUAL 95% CI'S FOR MEAN BASED ON POOLED STDEV			
(-----*-----)			
(-----*-----)			

MTB &gt; Oneway 'RSS (%)' 'source'.

SOURCE	DF	SS	MS
source	1	40.6	40.6
ERROR	6	113.9	19.0
TOTAL	7	154.6	

F	p
2.14	0.194

INDIVIDUAL 95% CI'S FOR MEAN  
BASED ON POOLED STDEV

LEVEL	N	MEAN	STDEV
1	4	65.607	4.302
2	4	70.115	4.412

POOLED STDEV = 4.358

65.0	70.0	75.0
INDIVIDUAL 95% CI'S FOR MEAN BASED ON POOLED STDEV		
(-----*-----)		
(-----*-----)		



## APPENDIX C-1

**SCOD removal efficiencies at various retention times for unseeded wastewater at 20°C.**

WWTP	SS <sub>0</sub>	SCOD <sub>0</sub>	RSCOD 1hour	RSCOD 2 hours	RSCOD 3hours	RSCOD 4hours	RSCOD 6hours	RSCOD 8hours
Sacriston	230	275	15	10	25	40	80	110
Sacriston	380	380	20	45	65	80	105	120
Sacriston	380	390	50	65	90	115	155	190
Sacriston	400	385	30	50	65	95	130	145
Sacriston	310	375	30	60	50	80	125	155
Sacriston	500	365	25	30	60	65	125	145
Sacriston	470	300	30	50	80	100	100	110
Sacriston	310	290	20	45	65	75	135	135
Sacriston	380	340	40	60	70	95	120	130
Sacriston	240	305	15	40	55	80	105	110
Sacriston	280	255	20	30	40	65	90	100
Sacriston	270	290	40	50	80	95	105	115
Durham	540	250	65	85	95	120	125	165
Durham	660	320	75	80	125	140	155	195
Durham	640	310	60	100	85	130	150	195
Durham	630	295	90	105	140	150	155	190
Durham	740	315	80	105	135	140	160	185
Durham	580	350	20	90	130	170	190	230
Durham	550	300	55	95	115	90	140	145
Broomhaugh	420	160	70	60	75	75	90	85
Broomhaugh	340	215	35	25	65	20	30	95
Broomhaugh	300	175	20	40	50	65	70	95
Broomhaugh	160	70	-5	10	15	10	15	15
Broomhaugh	330	200	45	60	75	80	80	100
Broomhaugh	410	155	35	45	45	60	70	80
Broomhaugh	400	185	10	30	35	30	55	70
Broomhaugh	270	165	15	25	35	45	55	75

Notes: (i) all units in mg/l

(ii) RSCOD = SCOD removal after a specified retention period

## APPENDIX C-2

### Results of regression Analysis of the data shown in Appendix C-1

Notes: A variable is not significant if  $p > 0.05$

#### (a) Retention time = 1 hour

The regression equation is:  $RSCOD = -9.0 + 0.126 SS_0 - 0.0204 SCOD_0$

Predictor	Coef	Stdev	t-ratio	p
Constant	-8.96	12.00	-0.75	0.463
$SS_0$	0.12571	0.02286	5.50	0.000
$SCOD_0$	-0.02036	0.04064	-0.50	0.621

$s = 16.09$   $R\text{-sq} = 57.9\%$   $R\text{-sq}(\text{adj}) = 54.4\%$

#### (b) Retention time = 2 hours

The regression equation is :  $RSCOD = -16.6 + 0.151 SS_0 + 0.0353 SCOD_0$

Predictor	Coef	Stdev	t-ratio	p
Constant	-16.58	11.32	-1.46	0.156
$SS_0$	0.15070	0.02156	6.99	0.000
$SCOD_0$	0.03532	0.03833	0.92	0.366

$s = 15.17$   $R\text{-sq} = 72.6\%$   $R\text{-sq}(\text{adj}) = 70.3\%$

#### (c) Retention time = 3 hours

The regression equation is :  $RSCOD = -18.1 + 0.177 SS_0 + 0.0658 SCOD_0$

Predictor	Coef	Stdev	t-ratio	p
Constant	-18.11	13.37	-1.35	0.188
$SS_0$	0.17677	0.02546	6.94	0.000
$SCOD_0$	0.06584	0.04526	1.45	0.159

$s = 17.92$   $R\text{-sq} = 73.7\%$   $R\text{-sq}(\text{adj}) = 71.5\%$

#### (d) Retention time = 4 hours.

The regression equation is :  $RSCOD = -30.4 + 0.169 SS_0 + 0.169 SCOD_0$

Predictor	Coef	Stdev	t-ratio	p
Constant	-30.44	16.86	-1.81	0.083
$SS_0$	0.16880	0.03210	5.26	0.000
$SCOD_0$	0.16924	0.05708	2.97	0.007

$s = 22.59$   $R\text{-sq} = 70.0\%$   $R\text{-sq}(\text{adj}) = 67.5\%$

## APPENDIX C-2-continued

### Results of regression Analysis of the data shown in Appendix D-1

#### (e) Retention time = 6 hours

The regression equation is:  $\text{RSCOD} = -31.7 + 0.136 \text{ SS}_0 + 0.304 \text{ SCOD}_0$

Predictor	Coef	Stdev	t-ratio	p
Constant	-31.72	14.43	-2.20	0.038
$\text{SS}_0$	0.13648	0.02749	4.97	0.000
$\text{SCOD}_0$	0.30393	0.04887	6.22	0.000

$s = 19.34$      $\text{R-sq} = 80.7\%$      $\text{R-sq(adj)} = 79.1\%$

#### (f) Retention time = 8 hours

The regression equation is :  $\text{RSCOD} = -32.0 + 0.175 \text{ SS}_0 + 0.324 \text{ SCOD}_0$

Predictor	Coef	Stdev	t-ratio	p
Constant	-32.01	15.84	-2.02	0.055
$\text{SS}_0$	0.17477	0.03016	5.79	0.000
$\text{SCOD}_0$	0.32445	0.05362	6.05	0.000

$s = 21.22$      $\text{R-sq} = 82.4\%$      $\text{R-sq(adj)} = 80.9\%$

**APPENDIX C-3****Rates of Reaction (Benfield and Randall, 1985)****1- Zero-order reaction**

$$C - C_0 = - kt$$

**2- First-order reaction**

$$C = C_0 e^{-kt}$$

**3- Second-order reaction**

$$\frac{1}{C} - \frac{1}{C_0} = kt$$

Where:

$C$  = concentration of reactant at time  $t$

$C_0$  = the constant of integration which is calculated as  $C = C_0$  at  $t = 0$

$k$  = the reaction rate constant

## APPENDIX D-1

## SCOD Removal efficiencies at an added seed concentration range of 0-1000 mg/l

SS <sub>0</sub>	SCOD <sub>0</sub>	RSCOD 1 hour	RSCOD 2 hours	RSCOD 3 hours	RSCOD 4 hours	RSCOD 6 hours	RSCOD 8 hours	seed con. (B)
540	250	65	85	95	120	125	165	0
660	320	75	80	125	140	155	195	0
640	310	60	100	85	130	150	195	0
630	295	90	105	140	150	155	190	0
740	315	80	105	135	140	160	185	0
580	350	20	90	130	170	190	230	0
550	300	55	95	115	90	140	145	0
300	175	20	40	50	65	70	95	0
280	255	20	30	40	65	90	100	0
270	290	40	50	80	95	105	115	0
330	200	45	60	75	80	80	100	0
230	275	15	10	25	40	80	110	0
410	155	35	45	45	60	70	80	0
270	165	15	25	35	45	55	75	0
380	390	50	65	90	115	155	190	0
400	385	30	50	65	95	130	145	0
310	375	30	60	50	80	125	155	0
500	365	25	30	-	-	125	145	0
470	300	30	50	80	100	100	110	0
310	290	20	45	65	75	135	135	0
380	340	40	60	70	95	120	130	0
240	305	15	40	55	80	105	110	0
420	160	70	60	75	75	90	85	0
160	70	-5	10	15	10	15	15	0
400	185	10	30	35	30	55	70	0
340	215	35	25	65	20	30	95	0
740	310	120	150	155	155	200	210	100
580	340	50	120	170	170	210	255	100
550	300	75	115	125	150	180	180	100
390	180	30	45	50	65	75	95	100
390	370	50	50	85	120	145	180	100
380	380	90	110	160	180	215	245	100
350	210	70	95	95	110	120	130	100
470	290	90	85	135	135	170	185	100
440	305	70	90	115	110	145	160	100
740	310	115	140	175	185	215	225	250
580	340	135	165	175	210	200	240	250
550	300	85	125	140	160	195	185	250
390	180	55	70	80	105	100	105	250
390	370	70	95	150	170	185	215	250
380	380	90	150	170	205	220	260	250
350	210	105	115	125	125	135	135	250
470	290	125	140	160	150	175	180	250

## APPENDIX D-1-continued

## SCOD Removal efficiencies at an added seed concentration range of 0-1000 mg/l

SS0	SCOD0	RSCOD 1 hour	RSCOD 2 hours	RSCOD 3 hours	RSCOD 4 hours	RSCOD 6 hours	RSCOD 8 hours	seed con. (B)
440	300	120	130	160	145	185	180	250
400	355	70	135	170	200	225	230	500
310	355	150	180	190	220	240	255	500
500	325	125	160	185	180	230	225	500
350	205	120	125	125	125	125	143	500
470	250	120	140	150	155	160	165	500
440	300	130	160	170	170	200	180	500
400	345	105	160	200	210	230	250	750
310	345	180	180	225	225	240	255	750
500	305	140	160	185	190	205	215	750
350	200	120	120	120	120	125	130	750
440	295	155	175	160	180	185	170	750
290	310	180	210	210	195	210	210	750
680	350	195	215	225	235	230	350	1000
530	355	150	225	230	230	255	260	1000
560	195	*	115	115	120	120	125	1000
520	390	235	250	270	260	270	260	1000
520	350	205	240	235	245	250	250	1000
480	230	125	140	155	115	140	135	1000

Note: All units in mg/l.

## APPENDIX D-2

### Results of multiple regression analysis of the data shown in Appendix D-1

The regression equation is

$$\text{RSCOD} = -110 + 0.479 \text{ SCOD}_0 + 0.0457 \text{ SS}_0 + 17.2 t + 11.2 b - 1.11 bt$$

Predictor	Coef	Stdev	t-ratio	p
Constant	-109.65	12.76	-8.60	0.000
scod0	0.47904	0.02989	16.03	0.000
ss	0.04569	0.01650	2.77	0.006
t	17.170	1.514	11.34	0.000
b	11.156	1.085	10.28	0.000
bt	-1.1117	0.2359	-4.71	0.000

s = 26.40    R-sq = 76.7%    R-sq(adj) = 76.2%

### APPENDIX D-3

#### Results of multiple regression analysis of SOUR data (Figures 7.10-7.14)

##### (i) Seed concentration = 100 mg/l

The regression equation is

$$\text{SOUR} = 10.4 - 0.868 t + 0.0115 \text{SS}_0 - 0.0176 \text{SCOD}_0$$

Predictor	Coef	Stdev	t-ratio	p
Constant	10.390	5.880	1.77	0.087
t	-0.8682	0.4571	-1.90	0.067
SS <sub>0</sub>	0.011479	0.008585	1.34	0.191
SCOD <sub>0</sub>	-0.01758	0.01973	-0.89	0.380

$$s = 5.959 \quad R\text{-sq} = 15.2\% \quad R\text{-sq}(\text{adj}) = 7.3\%$$

##### (ii) Seed concentration = 250 mg/l

The regression equation is

$$\text{SOUR} = 7.01 - 1.63 t + 0.0006 \text{SCOD}_0 + 0.0134 \text{SS}_0$$

Predictor	Coef	Stdev	t-ratio	p
Constant	7.011	5.001	1.40	0.169
t	-1.6266	0.4643	-3.50	0.001
SCOD <sub>0</sub>	0.00060	0.01765	0.03	0.973
SS <sub>0</sub>	0.013443	0.008842	1.52	0.137

$$s = 6.267 \quad R\text{-sq} = 29.6\% \quad R\text{-sq}(\text{adj}) = 24.1\%$$

##### (iii) Seed concentration = 500 mg/l

The regression equation is

$$\text{SOUR} = 25.2 - 3.10 t - 0.0089 \text{SCOD}_0 - 0.0036 \text{SS}_0$$

Predictor	Coef	Stdev	t-ratio	p
Constant	25.19	10.85	2.32	0.031
t	-3.0982	0.6718	-4.61	0.000
SCOD <sub>0</sub>	-0.00894	0.02280	-0.39	0.699
SS <sub>0</sub>	-0.00362	0.02103	-0.17	0.865

$$s = 6.756 \quad R\text{-sq} = 51.9\% \quad R\text{-sq}(\text{adj}) = 44.7\%$$



### APPENDIX D-3-Continued

#### Results of multiple regression analysis of SOUR data (Figures 7.10-7.14)

(iv) Seed concentration = 750 mg/l

The regression equation is

$$\text{SOUR} = 28.1 - 4.43 t - 0.0017 \text{SCOD}_0 + 0.0016 \text{SS}_0$$

Predictor	Coef	Stdev	t-ratio	p
Constant	28.10	16.79	1.67	0.108
t	-4.4347	0.9793	-4.53	0.000
SCOD <sub>0</sub>	-0.00172	0.03893	-0.04	0.965
SS <sub>0</sub>	0.00159	0.03101	0.05	0.960

s = 11.38    R-sq = 48.5%    R-sq(adj) = 41.5%

(v) Seed concentration = 1000 mg/l

The regression equation is

$$\text{SOUR} = 41.0 + 0.0872 \text{SCOD}_0 - 0.0182 \text{SS}_0 - 8.27 t$$

Predictor	Coef	Stdev	t-ratio	p
Constant	41.03	25.00	1.64	0.111
SCOD <sub>0</sub>	0.08722	0.03131	2.79	0.009
SS <sub>0</sub>	-0.01817	0.04545	-0.40	0.692
t	-8.266	1.290	-6.41	0.000

s = 16.42    R-sq = 61.2%    R-sq(adj) = 57.3%

# APPENDIX E-1

## SCOD results for Greater Amman wastewater

t	SS <sub>0</sub>	SCOD <sub>0</sub>	RSCOD	b	bt
4	750	410	160	1.0	4
4	800	425	215	1.0	4
4	790	370	200	1.0	4
4	1060	375	155	1.0	4
4	750	400	220	2.0	8
4	800	415	215	2.0	8
4	790	360	210	2.0	8
4	1060	370	165	2.0	8
4	710	355	230	3.5	14
4	960	350	210	3.5	14
4	1030	350	170	3.5	14
4	720	310	190	3.5	14
4	710	330	210	6.0	24
4	960	335	239	6.0	24
4	1030	335	208	6.0	24
4	720	295	198	6.0	24
8	750	410	225	1.0	8
8	800	425	250	1.0	8
8	790	370	215	1.0	8
8	1060	375	250	1.0	8
8	750	400	270	2.0	16
8	800	415	265	2.0	16
8	790	360	240	2.0	16
8	1060	370	250	2.0	16
8	710	355	215	3.5	28
8	960	350	210	3.5	28
8	1030	350	195	3.5	28
8	720	310	200	3.5	28
8	710	330	215	6.0	48
8	960	335	225	6.0	48
8	1030	335	200	6.0	48
8	720	295	199	6.0	48

## APPENDIX E-2

### Regression analysis of data shown in Appendix E-1

The regression equation is:

$$\text{RSCOD} = -44.3 + 0.464 \text{ SCOD}_0 - 0.0342 \text{ SS}_0 + 16.7 t + 25.1 b - 3.20 bt$$

Predictor	Coef	Stdev	t-ratio	p
Constant	-44.29	70.08	-0.63	0.533
SCOD <sub>0</sub>	0.4645	0.1549	3.00	0.006
SS <sub>0</sub>	-0.03424	0.02572	-1.33	0.195
t	16.706	3.309	5.05	0.000
b	25.096	6.199	4.05	0.000
bt	-3.2010	0.9070	-3.53	0.002

s = 19.33    R-sq = 60.0%    R-sq(adj) = 52.3%