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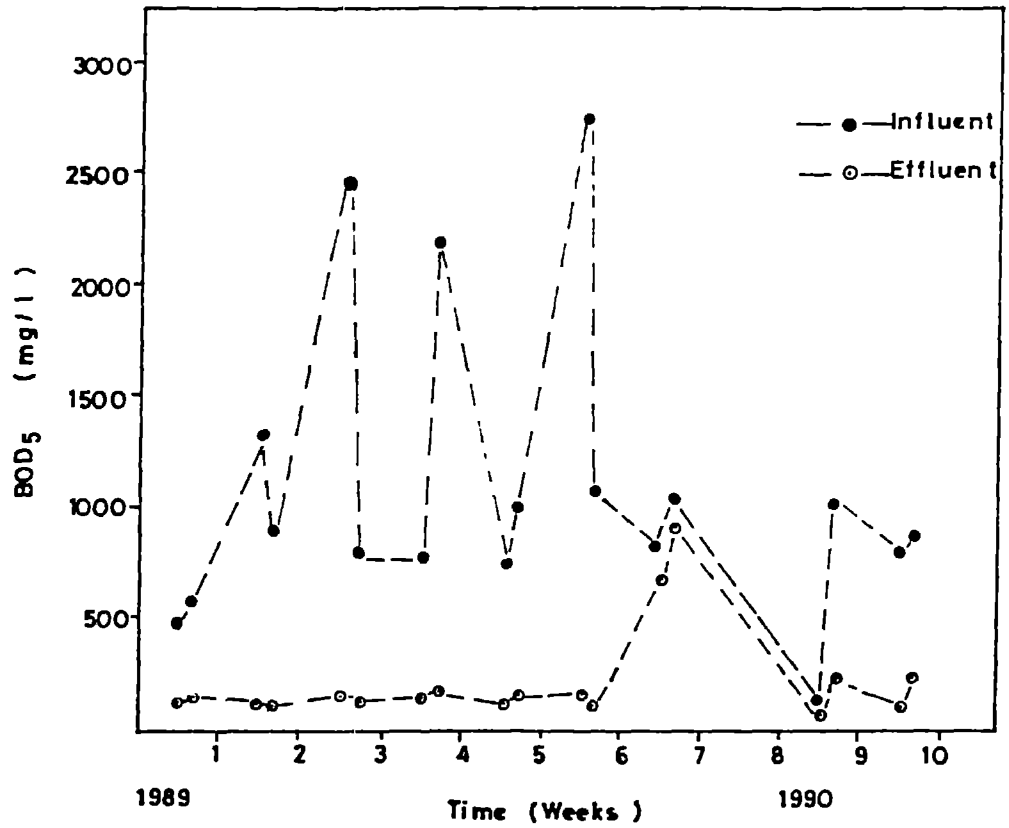

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EVALUATION OF OXIDATION DITCH TECHNOLOGY WITH REFERENCE TO THE LIMURU PLANT IN KENYA

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ABSTRACT

The performance of the oxidation ditch plant at Limuru was investigated for 10 weeks. Five-day biochemical oxygen demand (BOD_5) at 20°C, chemical oxygen demand (COD), suspended solids (SS) and fecal coliform organisms were analysed from the influent and effluent. Mixed liquor suspended solids (MLSS), mixed liquor volatile suspended solids (MLVSS), dissolved oxygen (DO), and 30 minute sludge settling volumes were determined for samples obtained from the oxidation ditch. Other parameters measured were pH and temperature for the influent, oxidation ditch mixed liquor, effluent and visibility depths in the settling tanks.

The major aim of the study was to find out whether the treatment plant was meeting the effluent discharge requirements as stipulated by pollution control authorities. The objectives of this study were:

1) to evaluate the performance of the treatment plant
2) to identify factors limiting plant performance and propose remedial measures.

In addition to analysis of samples, the following methods were used in the study:

1) extensive literature review
2) examination of available records
3) discussion with treatment plant staff
4) observations at the treatment plant.

The BOD_5 removal efficiencies were in the range 12 - 94 % with an average of 72 % and none of the effluent samples met the 20 mg/l BOD_5 limit. The corresponding figures for SS were 43 - 96 % and 74 % and none of the samples met the 30 mg/l limit. The visibility depths in the settling tanks averaged 23 cm and 24 cm.

Although the study was undertaken in only 10 weeks, there is a clear indication that the plant performance is not satisfactory and factors identified as contributing to the poor performance were inadequacies in design, operation, maintenance and management.

Process control at the plant is poor and the operators seem not to have the right concept of treatment processes. An educational campaign and on-the-job training are needed for the plant operators to control and operate the plant more efficiently. More intensive operation control and in-plant research should also be started. This will require better laboratory and backup facilities.

To overcome the problems of operation and maintenance (O&M) financing, appropriate effluent discharge tariffs should be introduced to make the plant self-supporting.

1 INTRODUCTION

The provision of sewerage systems on a large scale in Kenya was intended to improve and safeguard the hygiene and health conditions in the urban centres and to reduce the pollution of natural water courses (Danida 1982). One of the towns which benefited from the provision of a sewerage system was Limuru, which has about 7 000 inhabitants. Limuru is situated about 30 km north-west of Nairobi (Figure 1).

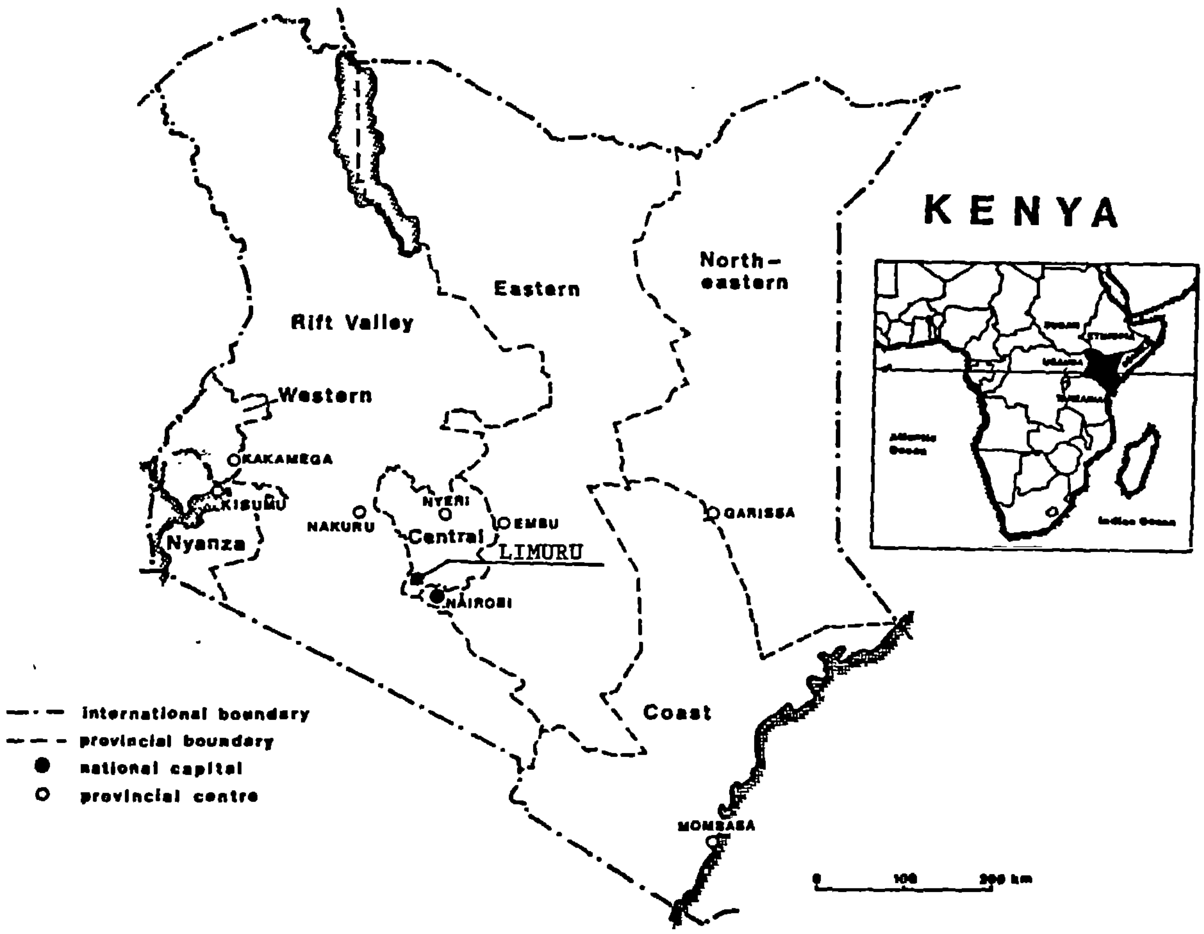

Figure 1. Location of Limuru town.

The town of Limuru lies at latitude 1° 7′ south and longitude 36° 38′ east.

The average elevation is about 2 250 m above sea level. The mean maximum temperature is 21°C and the mean minimum 11°C.

A study of the wastewater sources in 1989 showed that most of them are domestic with small quantities coming from the slaughterhouse and PVC pipe factory. There is a shoe factory which has its own wastewater treatment plant but discharges the effluent into the natural pond into which the municipal treatment plant also discharges its effluent.

Limuru was selected for the study because it is run by the Ministry of Water Development (MoWD) and is near laboratory facilities. The history of pollution from the shoe factory and the problems experienced at the treatment plant were other considerations.

2 OXIDATION DITCH TECHNOLOGY IN WASTEWATER TREATMENT

2.1 Process description, development and applications

2.1.1 Process description

The oxidation ditch is a modified form of the activated sludge process in which a continuous loop reactor is used as a bioreactor in which mixed liquor is recirculated continuously. The process normally used in oxidation ditches is extended aeration, i.e. long hydraulic and solids retention times and low organic loading rates. A controlled aeration and mixing device is used to aerate and circulate mixed liquor around the channel (Mandt and Bell 1982). In a typical municipal wastewater treatment application, primary settling is not needed before the oxidation ditch. Screening or comminution and grit removal are recommended. Secondary settling tanks are normally provided. An effluent chlorination unit is optional (Mandt and Bell 1982). Although essential, provision for improving the bacteriological quality of the effluent is seldom made (Mara 1978).

2.1.2 Development of the oxidation ditch technology

The oxidation ditch was developed during the 1950's at the Research Institute for Environmental Hygiene (TNO) in the Netherlands (Mandt and Bell 1982).

The process was developed as follows:

1953 The oxidation ditch was developed by Dr A. Pasveer from the TNO Research Institute (Heide 1982).

1954 The first full-scale plant was installed at Voorschoten, Holland to serve a population equivalent of 360, and it was intermittent flow type (Mandt and Bell 1982).

1962 First full-scale plant was installed at Williams Lake, B.C., Canada (Schmidtke 1982).

1967 LeCompt and Mandt developed the first application of a submerged aeration and circulation system known as the Jet Aeration Channel (LeCompt and Mandt 1971, cited by Mandt and Bell 1982).

1968 Dutch engineers working for DHV Consulting Engineers used slow-speed surface turbines placed at the end of a centre baffle to provide the circulation for the oxidation ditch. This is the Carrousel process (Mandt and Bell 1982).

Since the early developments of Pasveer, LeCompt, Mandt and DHV, numerous process and mechanical modifications have been applied to oxidation ditch channels. The applications of the oxidation ditch from the first track have expanded tremendously. The technology is now firmly established and accepted as a treatment technique (Mandt and Bell 1982). The acceptance is evident from the operation of more than 750 oxidation ditches in North America with 71 installations in eight of Canada's provinces (Schmidtke 1982). There are over 3 000 installations of the Pasveer-type ditches in Western Europe alone with about 200 Carrousels existing and several others under construction (Arceivala 1986).

2.1.3 Applications

All kinds of sewage of domestic or industrial origin containing biologically decomposable waste materials, and no or only small amounts of compounds which may disturb microbiological processes, can be treated by oxidation ditch principles. Sometimes, in sewage of industrial origin, there are specific problems related to the nature of the sewage, e.g. high concentration of heavy metals. These problems are not confined to oxidation ditches but affect all biological wastewater treatment systems. Owing to the large buffer capacity of the very low loaded activated sludge plants, the vulnerability to shock loads, or a restricted amount of toxic compounds is lower than in high loaded activated sludge systems (Heide 1982).

Oxidation ditch principles are applied in small units for domestic use as an alternative to septic tanks as well as large plants for exceeding 100 000 population equivalent. Examples of industries employing oxidation ditches for treating wastewater are dairy, brewing, textile, tanning, oil processing and chemicals (Heide 1982).

2.2 Main types of oxidation ditches

Pasveer and Carrousel types are the oxidation ditches widely used.

2.2.1 Pasveer ditches

Pasveer oxidation ditches (Figure 2) are often preferred for small installations. The ditch is used on a discontinuous basis. With the discontinuous system, the settling, inflow, discharge and aeration periods are intermittent and follow a specific cycle of operation, with the ditch itself serving as a clarifier during the quiescent period of the cycle (Briscoe 1967).

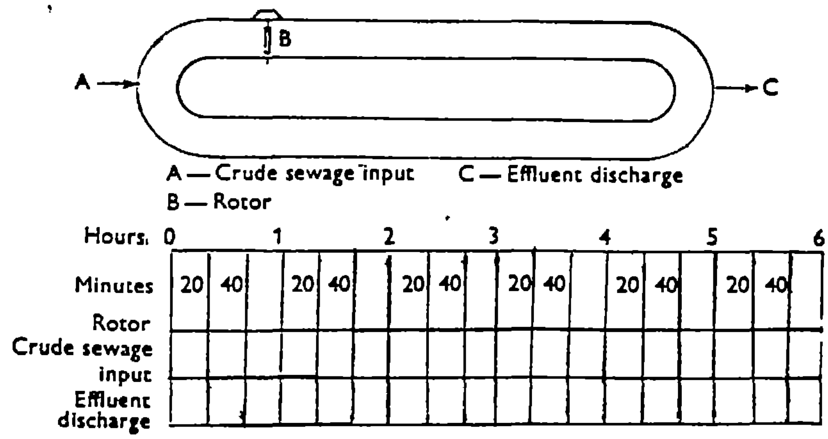

Figure 2. Pasveer oxidation ditch, operated on a discontinuous basis (Briscoe 1967).

The intermittent operation gives good BOD and nitrogen removal, but requires that the raw wastewater is kept in sewers and pump sump when aeration is stopped (Arceivala 1981). To avoid anaerobic conditions, especially if flows are high and the climate is warm, Arceivala (1981) recommended continuous operation of the ditch. This can be achieved by

1. providing two discontinuous ditches in parallel so that the continuous inflows can be diverted from one to the other as required

2. providing an arm in the ditch which can be isolated for use as a settling compartment without disturbing the rest of the ditch (Figure 3).

3. providing a separate settling tank (Figure 4).

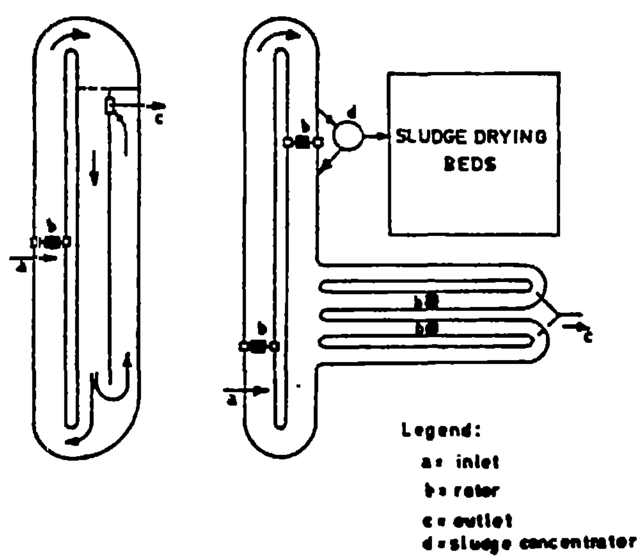

Figure 3. Continuously operated oxidation ditch with arm (Arceivala 1981).

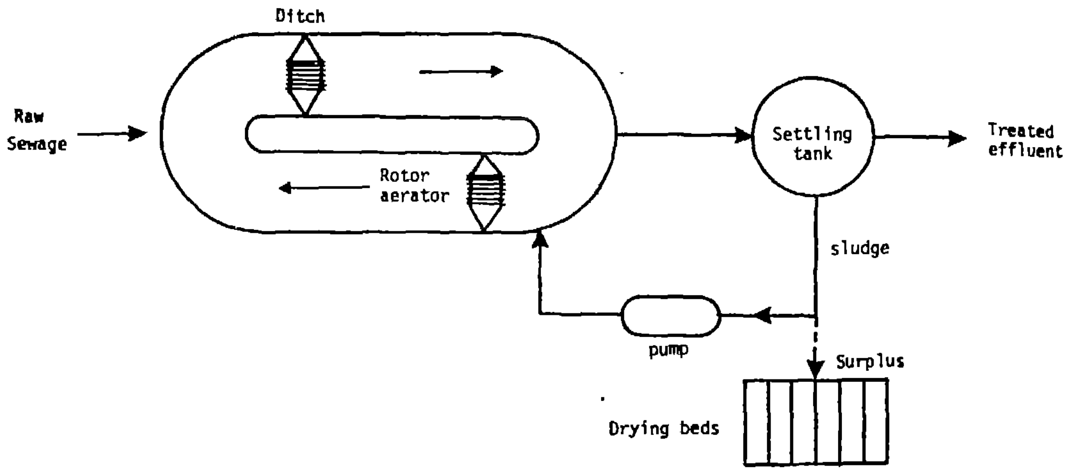

Figure 4. Continuously operated oxidation ditch with conventional settling tank (Cairncross and Feachem 1983).

The first two alternatives often require installation of automatic time switches to start and stop aeration and to open and close certain inflow and outflow valves. This may not be a suitable alternative in some developing countries, and the provision of a separate settling tank may be the preferred alternative even for small installations (Arceivala 1981).

Pasveer oxidation ditches are about 1.0 - 1.5 m deep and are generally aerated by horizontal axis mechanical rotors which create sufficient velocities (0.3 m/s or more) to keep all solids in suspension (Mara 1978).

Construction of Pasveer oxidation ditches may be either in earthwork with sloping embankments or in brick or stone masonry with vertical walls. The ditch should preferably have a concrete lining with slopes of about 1:1.5. The following modifications can be adopted (Mara 1978).

1. A lining of butyl rubber instead of concrete although this is not as long lasting as concrete.

2. Rigid lining of concrete or concrete slabs to be placed under the rotors and for a distance 5 m downstream to prevent damage due to the high turbulence in these areas if a flexible lining is used.

3. When mammoth rotors are used, the ditch should be 3 - 5 m deep with vertical reinforced concrete walls.

4. When cage rotors are used the depth of the ditch should be 1.0 - 1.5 m.

2.2.2 Carrousel ditches

Carrousel oxidation ditch is a development of the Pasveer ditch. Aeration is achieved by fixed surface aerators instead of cage rotors. The surface aerators are normally placed at the end of the ditch and the water depth is 2 - 4 m (Mara 1978).

Figure 5 shows a typical Carrousel oxidation ditch.

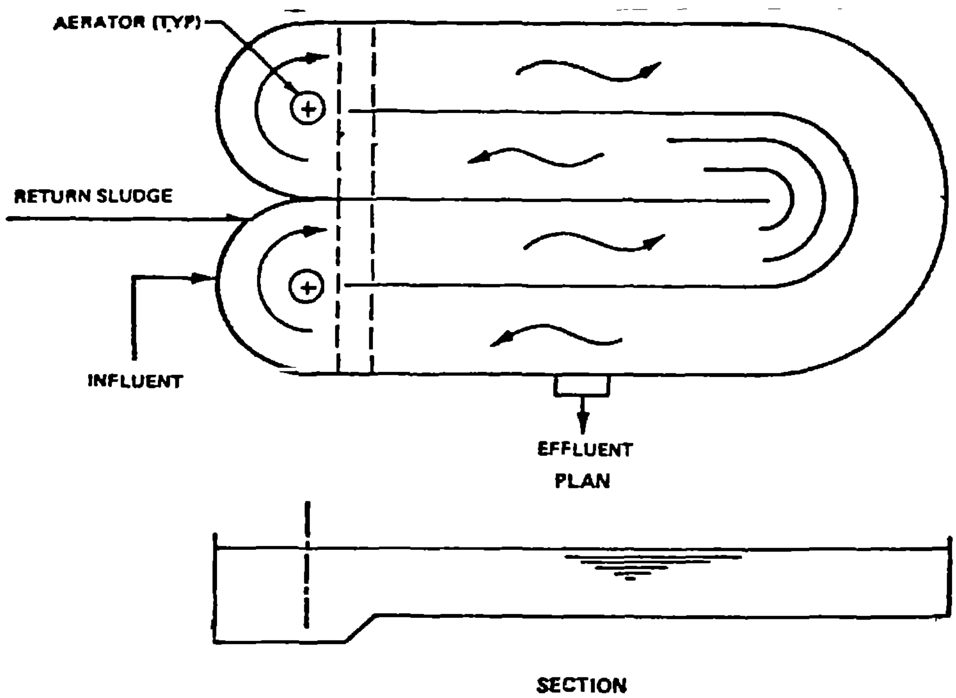

Figure 5. Typical Carrousel oxidation ditch (Jacobs 1975).

Aeration is concentrated at one point, instead of being distributed over the length of the ditch (Jacobs 1975). The mixed liquor is rich in oxygen as it flows out of the aeration zone, but low in oxygen as it returns to the zone. This enhances oxygen transfer. Separate settling tanks are generally provided (Arceivala 1981).

Figure 6 shows a configuration highly recommended, because it incorporates nitrification and denitrification besides BOD removal.

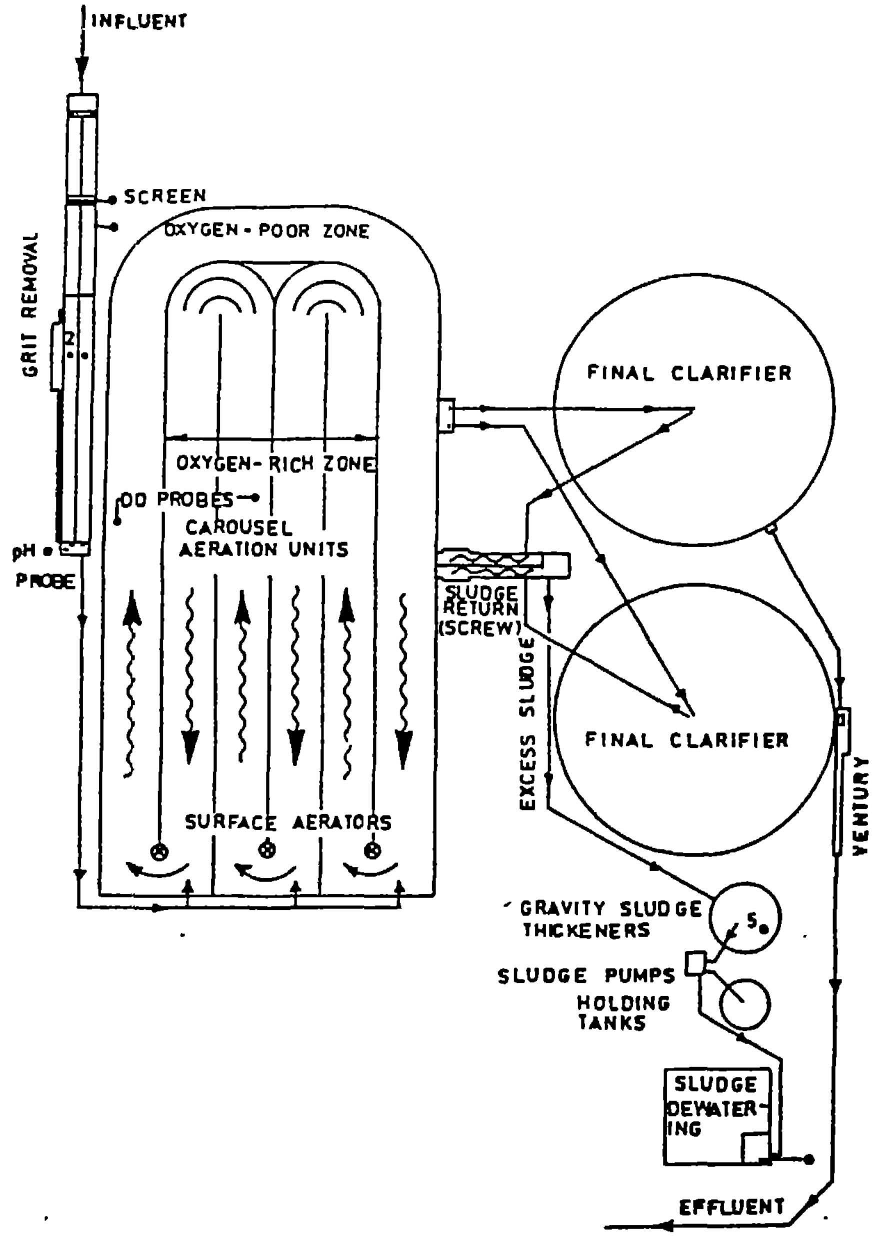

Figure 6. Carrousel system for nitrification, denitrification and BOD removal (Arceivala 1981).

The channel section where denitrification takes place is maintained with a DO concentration of 0.5 mg/l or less, and a part of the influent is diverted to this section to serve as a carbon source for denitrification. The other sections of the ditch have a DO content of about 2 mg/l and, with sufficient mean cell residence time, nitrification and denitrification occur. Although the receiving water body may not require a denitrified effluent, it is advisable to incorporate denitrification in the design of the plant as an energy conservation measure (Arceivala 1981).

The walls of Carrousel ditches are constructed of reinforced concrete (Mara 1978).

2.2.3 Comparison of the Pasveer and Carrousel ditch systems

Pasveer and Carrousel ditch systems, although both operate on the extended aeration mode, have numerous basic differences (Table 1).

Table 1. Comparison of Pasveer and Carrousel ditch systems (Mulready et al 1982).

	Pasveer	Carrousel
Type of aerator	The TNO rotor consists of a series of rows of blades arranged around a horizontal centre shaft	Vertical aerators of several types have been used. The most common is the Simcar type with about eight vanes projecting from a centre shroud
Depth of immersion	Controlled by adjusting the level of the outlet weir from the oxidation ditch	Usually controlled by a jacking screw on the vertical shaft. Future plants may use the adjustable outlet weir for level control
Siting of aerator	The aerator may be sited in any position along the ditch	The surface aerator must be positioned above the deep section of the ditch
Construction	Uniform continuous channel usually 1.0-1.8 m deep	Uniform channel usually 2 m in depth with aeration pockets 3 m deep

Oxidation ditches have two main functions: first to supply the required oxygen for micro-organisms, and secondly to set up a stirring motion to ensure contact between the micro-organisms, the influent and the oxygen introduced (Mulready et al 1982).

This is the similarity regardless of the method of aeration used. Comparison between the two systems can be made with reference to (Mulready et al 1982):

1. oxygen supply capacity
2. efficiency of mixing
3. adaptation
    (a) to different oxygen requirements
    (b) to the production of anoxic zones
    (c) for different depths of aerator immersion
4. reliability of all components
5. construction, maintenance and operation costs.

Factors affecting overall efficiency of the plant include (Mulready et al 1982):

1. distribution of the incoming flow to the plant

2. sludge produced within the plant

3. settling characteristics and general appearance of the sludge

4. variation in load

5. variation in flow

In a study comparing the two systems within the Bristol Avon Division of the Wessex Water Authority, Mulready et al (1982) concluded the following:

1. Operationally there is minimal difference between the two systems within the range considered.

2. Carrousel system is more expensive to construct than Pasveer ditch in the range of plant sizes considered. However when considering larger plants the saving in land may off-set the additional costs spent on the Carrousel.

3. Carrousel system appears to be more adaptable in the use of anoxic zones and effective DO control than Pasveer ditch.

4. Expansion of Pasveer system appears easier than Carrousel.

5. Generally Carrousel system produces better and higher quality effluent than Pasveer system.

2.3 Design criteria of oxidation ditch systems

There are various approaches to the design of oxidation ditches (Arceivala 1981, Benefield and Randall 1985). Essential items which can serve as a checklist for detailed design are presented below.

2.3.1 Primary treatment

Oxidation ditch plants require the same treatment facilities as other treatment systems. Pumping of influent is often required. If the collection system is long, resulting in long detention times, the wastewater may arrive at the treatment plant in a septic condition. Preaeration may be used in these cases for odour control and to prevent the formation of corrosive atmospheres (Mandt and Bell 1982).

Coarse screening is included in most treatment facilities to remove large particles which may damage equipment or piping. In medium to large plants, the bar screens are usually mechanically cleaned, whereas in smaller plants, they may be mechanically or manually cleaned (Mandt and Bell 1982).

Grit removal may be included in oxidation ditch plants. In general grit removal is carried out to protect equipment and piping from abrasion and clogging. The cost of grit removal facilities and their operation must be considered against the cost of grit resistant pumps and increased operation and maintenance costs due to abrasion and clogging (Mandt and Bell 1982).

Primary clarification is rarely used with oxidation ditches in the treatment of municipal wastewaters. When an oxidation ditch is to be designed in the conventional activated sludge range or when the wastewater has a very high solids concentration, primary clarifiers may be used and grit removal and sludge handling alternatives should be carefully considered (Mandt and Bell 1982).

2.3.2 Aeration tank

Important parameters for dimensioning the aeration tank include:

1. MLSS or MLVSS

2. mean cell residence time

3. hydraulic detention time

4. food/micro-organism (F/M) ratio.

About 54 g BOD is produced per capita per day. With the required sludge load of 0.05 kg BOD_5/kg sludge/d, 1 kg of sludge has to be present in an oxidation ditch. For an average MLSS concentration of 4 000 mg/l, one cubic metre of aeration volume is sufficient for a population equivalent of 4. The sludge load is the basic criterium of oxidation ditches. The hydraulic detention time is a derived parameter which is determined by the supply of water per population equivalent (Heide 1982). Heide (1982) states that a mean cell residence time of 25 days corresponds with a temperature > 8 °C for a sludge load of 0.05 kg BOD_5/kg sludge/d and recommends that for proper sludge stabilization a high sludge load of 0.07 - 0.1 kg BOD_5/kg sludge/d should be considered at higher temperatures as a precautionary measure. Heide (1982) points out that the temperature profile throughout the year should be considered and the lowest values adopted for design. Typical design values are given in Table 2.

Table 2. Typical design criteria for extended aeration (including Pasveer and Carrousel type ditches) (Arceivala 1981, modified by the author).

Parameter	Unit	Range
Mean cell residence time	d	
Warm (around 20°C or higher throughout)		10 - 20
Temperate (around 10 - 15°C in winter)		20 - 30
Cold (5 - 7°C or lower in winter)		> 30
F/M ratio	kg BOD_5/ kg MLVSS/d	
Warm		0.2 - 0.25
Temperate		0.1 - 0.2
Cold		$\leq$ 0.2
Hydraulic retention time	h	12 - 36
MLSS	mg/l	4000 - 5000
VSS/SS		0.5 - 0.8

*VSS - Volatile suspended solids

The shape of the ditch is generally an oval, but may be bent at one end, both ends, circular, or any other shape as long as it forms a complete circuit (Parker 1975). The geometric configuration and the power input per unit volume are selected to achieve the best possible mixing conditions (Arceivala 1986).

2.3.3 Aeration systems

The first step when dimensioning an aeration system is to determine the oxygen demand. Empirical approximations to oxygen demand are possible. Factors which relate the amount of oxygen (per kg) required per kg of BOD_5 applied or removed can be used. The factors or ratios are functions of the particular biological treatment system considered (Mandt and Bell 1982). Arceivala and Alagarsamy (1970), cited by Mara (1978), gave the aeration requirement (kg O_2/kg BOD_5) as 1.5 - 2.0 for India and 2.0 for Europe.

In nitrifying and denitrifying oxidation ditch systems, Mandt and Bell (1982) use a procedure that takes into account the oxygen required for BOD removal and nitrification as well as the oxygen available from denitrification.

Commonly used aeration systems are (Heide 1982):

1. horizontal rotors (surface aeration)

2. vertical rotors (surface aeration)

3. diffused air aeration (subsurface aeration).

Examples of horizontal rotors are the TNO rotor (cage rotor) and the mammoth rotor (Figures 7 and 8). The cage rotor is a brush aerator which superseded the Kessener brush. For bigger plants, i.e. plants serving a population above 10 000 the cage rotor has been superseded by the mammoth rotor (Scott and Smith 1980).

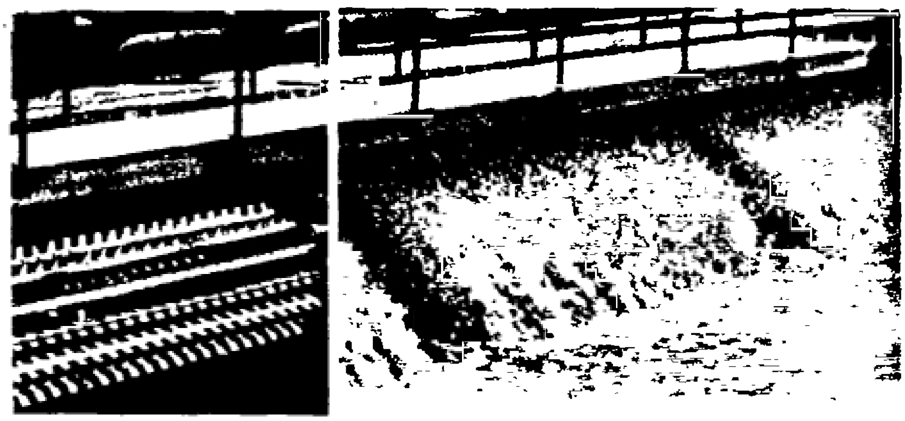

Figure 7. Cage rotor (Mara 1978).

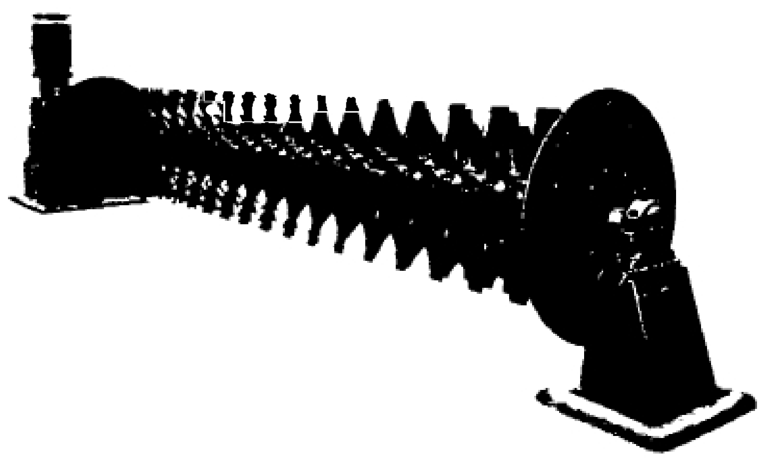

Figure 8. Mammoth rotor (Mara 1978).

Carrousel system uses vertical aeration rotors like Simcar (Heide 1982). Figures 9 and 10 show details of vertical axis aerators.

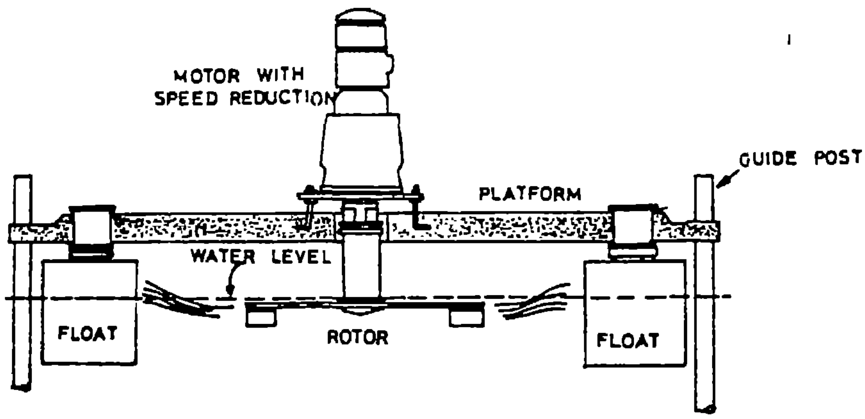

Figure 9. Detail of a typical low-speed vertical axis aerator (Arceivala 1981).

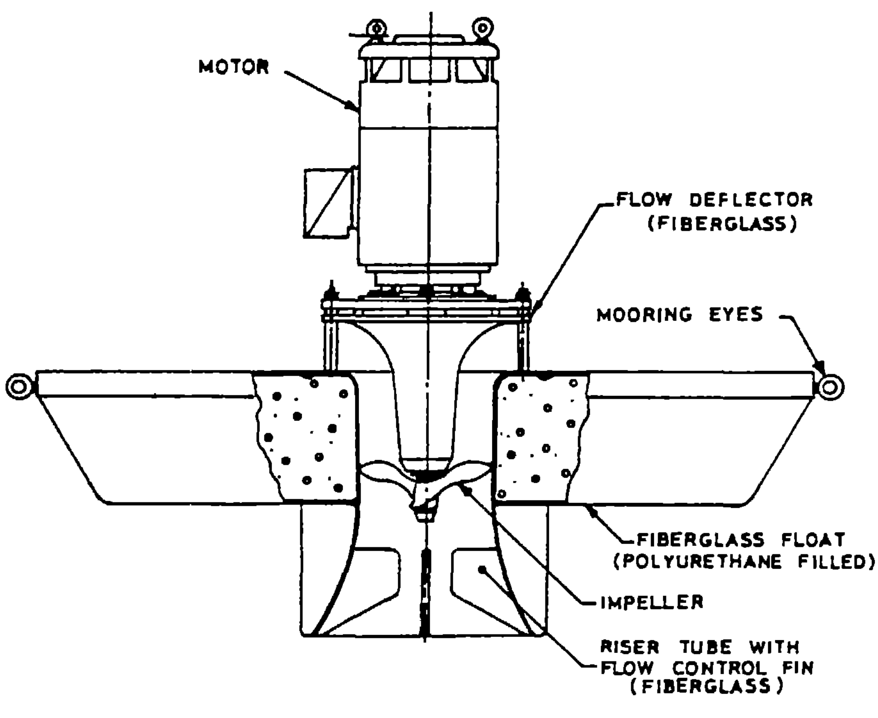

Figure 10. Detail of a floating high-speed vertical axis aerator (Arceivala 1981).

Diffused air aeration can also be applied in oxidation ditches. For example, in Australia deep rectangular tank arrangements use diffused air aeration (Heide 1982). An illustration of diffused air supply is given in Figure 11.

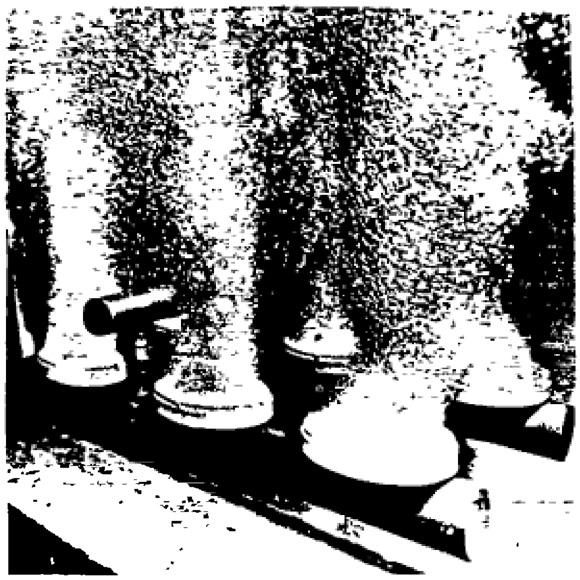

Figure 11. Porous plates for diffused air supply (Mara 1978).

In the use of diffused air aeration, the diffuser design determines bubble size, which, in turn categorizes diffusers as fine, medium, or coarse bubble devices (Gibbon 1974, Metcalf and Eddy 1979). Whether to use a fine, medium or coarse bubble devices is a design problem. Many manufacturers can show successful installations where the diffuser has been properly chosen to fit the requirements of the process and to integrate with the other components of the system (Gibbon 1974).

2.3.4 Settling tank

Dick (1976) points out that a settling tank must perform two functions:

1. clarification to obtain a clear effluent

2. thickening of the return sludge.

A basic understanding of the performance of the final settling tank in the activated sludge process is needed to permit optimization in treatment plant design and precise control of activated sludge treatment facilities (Dick 1976).

According to Dick (1976) the traditional approaches to dimensioning settling tanks were based on assumptions that did not adequately describe process performance. Dick (1976) concluded that the following parameters were not suitable for designing settling tanks:

1. limit to surface settling rate

2. minimum detention period

3. the relationship between the return sludge concentration and the sludge volume index (SVI)

4. quick recycling of the activated sludge solids

5. limit on solids loading on final settling tanks

6. the requirement that the overflow rate should equal the settling velocity of the mixed liquor.

Dick (1976) reviewed a procedure to settling tank design using the solids flux theory and concluded that this rational approach is useful for evaluating final settling tank design and operation.

Several researchers (Munch and FitzPatrick 1978, Pitman 1980, Anderson 1981, Johnstone et al 1979, Wilson and Lee 1982) agree with Dick (1976). Good agreement has been reported between laboratory experiments and observed performance in pilot scale equipment (Dick 1974, cited by White 1976). However, large deviations between laboratory experiments and full scale performance have also been reported (Hibberd 1974, cited by White 1976).

Mandt and Bell (1982) emphasize the need for pilot scale studies to collect sufficient data to permit settling tank dimensioning based on solids flux analysis or other theoretical considerations. But since it is often difficult to carry out pilot scale studies Mandt and Bell (1982) give the following design parameters for settling tanks (Table 3).

Table 3. Recommended settling tank design parameters for extended aeration (Mandt and Bell 1982, modified by the author).

Parameter	Unit	Recommended value
Overflow rate	$m^3/m^2/d$	12 - 20
Solids loading rate	$kg/m^2/d$	20 - 98
Weir loading rate	$m^3/m/d$	124 - 186

2.3.5 Return sludge

The purpose of the return sludge is to maintain a sufficient concentration of activated sludge in the aeration tank. It is essential in the process to return sludge from the settling tank to the inlet of the aeration tank (Metcalf and Eddy 1979).

Sufficient concentration of activated sludge in the aeration tank is essential. Table 4 shows some of the values recommended for design.

Table 4. Recommended activated sludge concentration in the aeration tank.

Source	MLSS mg/l
Pfeffer (1966)	5000 - 6000
Parker (1975)	3000 - 8000
Water Pollution Control Federation Manual of Practice No. 8, cited by Gemmell and Herbert (1985)	2000 - 6000
Metcalf and Eddy (1979)	3000 - 6000
Mara (1978), Heide (1982)	4000
Mandt and Bell (1982)	3500 - 5000
Arceivala (1986)	4000 - 5000

Adequate return sludge pump capacities must be provided to avoid the sludge blanket filling the entire depth of the tank at peak flows (Metcalf and Eddy 1979). Table 5 shows return sludge ratios reported by others.

Table 5. Recommended return sludge rate.

Source	Return sludge % of influent flow
Metcalf and Eddy (1979)	50 - 100 for big plants; up to 150 for small plants
Arceivala (1986)	75 - 100
Benefield and Randall (1985)	75 - 100
Mandt and Bell (1982)	50 - 100

2.3.6 Excess sludge

Excess sludge may be removed from the settling tank sludge being recycled, or from the mixed liquor before settling. The latter has certain advantages, and permits better control of the mean cell residence time (Arceivala 1986).

Thickening, drying beds, belt presses, filter presses and centrifuges are commonly used to dewater sludge (Heide 1982). In general it is observed that sludges from oxidation ditches are more difficult to dewater, for example using belt presses than anaerobically digested sludges (Heide et al 1982, cited by Heide 1982).

The dewatered sludge is spread on land where land is not a problem or used for landfilling (Mandt and Bell 1982). If used on agricultural land the concentration of heavy metals should be determined.

Sludge drying beds can be dimensioned according to the criteria given in Table 6.

Table 6. Area requirements for open sand beds for sludge drying (Arceivala 1981).

Type of sludge and region	Area m^2/person
Oxidation ditch sludge (Netherlands)	0.16 - 0.38
Extended aeration excess sludge (South Africa)	0.03 - 0.04
Oxidation ditch sludge and conventional digested sludge (India)	0.05 - 0.10
Conventional digested sludge (USA)	0.11 - 0.28

2.3.7 Nitrification and denitrification

Biological processes for nitrification with denitrification are becoming increasingly important for wastewater treatment. The addition of denitrification to a nitrification process can have several advantages over nitrification alone (Rittmann and Langeland 1985):

1. complete nitrogen removal

2. enhanced phosphorus removal

3. reduced aeration requirements

4. reduced alkalinity consumption

Reduced aeration requirement has been of the most widespread interest (Arceivala 1986, Heide 1982, Rittmann and Langeland 1985). Only a few facilities are required to remove nitrogen and phosphorus, and alkalinity consumption is not a serious concern in wastewaters with high alkalinity. On the other hand, reduced energy requirements which save energy costs and increase the capacity of an overloaded facility are of universal interest (Rittmann and Langeland 1985).

Heide (1982) states that savings in energy are important since energy consumption in oxidation ditches is high compared to that in stabilization ponds. According to Heide (1982), the main reason why in many cases denitrification is included as an integral part of biological treatment in very low loaded, activated sludge plants in the Netherlands is to reduce aeration requirements.

Current designs require provision of aerobic and anoxic zones for nitrification and denitrification respectively (Arceivala 1976, Forster et al 1984, Heide 1982). Rittmann and Langeland (1985) studied two full scale oxidation ditches and showed that anoxic zones within the reactor were not necessary and were never detected at any of the plants.

2.3.8 Phosphorus removal

The absence of presedimentation in the oxidation ditch process does not lead to a difference in removal of phosphorus compared to the conventional activated sludge plants or trickling filters. Depending on the P-content of the influent 30 - 40 % removal can be achieved (Heide 1982).

Sometimes additional removal of phosphorus is necessary and precipitation is used. In simultaneous precipitation a mixture of chemicals and biological sludge is present in the aeration tank. If postprecipitation is applied lower P-contents can be obtained. However, this procedure is expensive because an extra tank for sedimentation of chemical sludge is required. Preprecipitation is principally not possible in an oxidation ditch system because of the absence of a primary settling tank (Heide 1982).

Viraraghaven et al (1979) achieved 92 % phosphates (as P) reduction using liquid alum. Maiti et al (1988) used activated algae in an oxidation ditch model for the removal of nitrogen and phosphorous and achieved removals of 67 - 96 % for both nutrients.

2.4 Operation of oxidation ditches

2.4.1 Control of oxidation ditches

The overall control of an oxidation ditch plant depends on biomass control and control over DO. To achieve stable operation it is necessary to maintain a constant biodegradable organic load. The most common way of achieving this end is through control of the F/M ratio or solids retention time. Both parameters may be adjusted as required to meet changes in wastewater characteristics or temperature (Mandt and Bell 1982). Use of the F/M ratio requires knowledge of the biomass present. Real-time data regarding biodegradable organic loads are not available although this information can be approximated through the use of COD or total organic carbon (TOC) combined with BOD/COD or BOD/TOC correlations. In the USA where MLVSS is used instead of MLSS the biomass is traditionally measured by assuming VSS represent biomass. In oxidation ditch plants primary settling tanks are rarely used. Therefore, a significant but variable fraction of the MLSS measured may be nonviable organics. The need for real-time data and the possibility to measure nonviable matter make control based on F/M ratio difficult (Mandt and Bell 1982).

Control through solids retention time requires removing of a constant percentage of the biomass every day (Mandt and Bell 1982, Mara 1978). For example, to obtain a solids retention time of 20 days it is necessary to remove 5 % of the biomass daily. Since the viable biomass will be removed as a fixed fraction of the total solids removed, control may be achieved simply by removing the desired percentage of total ditch suspended solids. Control of oxidation ditches using solids retention time is recommended because it does not require real-time data (Mandt and Bell 1982).

Control of excess sludge requires facilities to measure its flow rate and concentration. Removal of sludge directly from the oxidation ditch offers advantages in control and further handling of the excess sludge (Metcalf and Eddy 1979, cited by Mandt and Bell 1982).

Return sludge must be controlled to maintain the desired solids content in the oxidation ditch. Continuous withdrawal of sludge from the settling tanks is recommended to prevent anoxic conditions in the sludge and the potential for rising sludge due to denitrification (Mandt and Bell 1982).

The other major control parameter for oxidation ditch plants is the DO concentration. Almost all oxidation ditch plants are designed to allow some control over oxygen input. This may be accomplished with variable immersion or speed on mechanical aerators and blower turndown when diffused aeration systems are used. In plants designed for BOD removal and/or nitrification control is utilized to maintain a DO level greater than or equal to 2 mg/l. Considerable cost savings through reduced aeration may be obtained by preventing unnecessarily high levels (Mandt and Bell 1982)

When denitrification is desired, close control over oxygen input is required to ensure maintenance of anoxic zones (Mandt and Bell 1982).

2.4.2 Operational problems with oxidation ditches

Common problems in oxidation ditch plants may be associated with the activated sludge, aeration, returned sludge and settling (Pfeffer 1966).

1. Activated sludge

   One of the most pronounced problems with oxidation ditches is the rising sludge in the final settling tanks. This is detrimental to treatment efficiency as these solids are carried out in the effluent. The problem results from solids deposited in the settling tanks. The gases produced by the solids raise the mass of sludge to the surface. The problem is enhanced by a high degree of denitrification (Pfeffer 1966).

   Bulking may be described as a phenomenon in activated sludge plants in which the sludge settles so slowly that the volume occupied by the sludge becomes excessive, the sludge volume builds up eventually to spill over with the settling tank overflow resulting in high SS concentrations in the effluent (Blackbeard et al 1986).

   Sludge bulking arises principally from the following two causes:

   1. presence of excessive numbers of filamentous micro-organisms, called filamentous bulking

   2. the absence of filamentous micro-organisms.

   Bulking attributable to excessive numbers of micro-organisms, is by far the most common (Blackbeard et al 1986).

   Filamentous micro-organisms are an essential part of the floc population. The filaments form the backbone to which floc forming bacteria adhere (Sezgin et al 1978, cited by Blackbeard et al 1986). If there are insufficient filaments, the floc is weak and subject to disintegration in the turbulent environment of the aeration zones, in this case fine floc particles are

present in the supernatant termed pin-point floc. If the filaments are very many, the sludge flocs are bound together by the filaments causing very poor settling sludge. A balance between floc formers and filamentous micro-organisms is desirable to yield a sludge with good settling and clarification properties (Blackbeard et al 1986).

Several researchers (Jenkins et al 1984, Strom and Jenkins 1984, Jones and Franklin 1985, Wanner et al 1987) have carried out extensive studies to identify some of the filamentous micro-organisms and possible control measures. Eikelboom (1982) states that one of the best available control measures is a high F/M ratio during the mixing of sewage and sludge.

2. Return sludge

Significant operational problems can also result from the sludge return system. In many plants the pumps cannot be controlled to return the sludge at the desired rate. Clogging of return lines is a common problem.

Foaming can be a problem if the concentration of MLSS is low. However, with 5 000 - 6 000 mg/l of SS, foaming is not usually a problem (Pfeffer 1966).

3. Settling

The settling efficiency of the secondary settling tank can be reduced by hydraulic surges resulting from pumping stations or storm water. The capacity of the pumps should be as close to the average flow rate through the plant as possible (Pfeffer 1966).

4. Aeration

Cold climates often cause operational problems when mechanical aerators are used. Instead of diffusing relatively warm air from a compressor into the liquid, the liquid is thrown out into the cold atmosphere with mechanical aeration, unless housed (Pfeffer 1966).

3 INVESTIGATION OF THE LIMURU OXIDATION DITCH PLANT

3.1 Description of wastewater treatment plant

The existing plant was designed to serve a population of 4 500 (population equivalent of 6 100). Phase II was dimensioned to increase the capacity of the treatment plant to serve a population of 11 000 (population equivalent of 14 300) (Gauff... 1976).

The hydraulic load was based on a dry season flow of 550 m^3/d and a peak wet season flow of 7 340 m^3/d for the existing plant and these figures were to be increased to 1 130 m^3/d and 13 100 m^3/d after completion of Phase II (Gauff... 1976).

The existing plant was designed to handle 330 kg BOD_5/d with provision for increasing the capacity to 790 kg/d when Phase II is constructed. The corresponding figures for SS are 440 kg/d and 1 030 kg/d respectively (Gauff... 1976).

Examination of the BOD_5 and SS shows concentrations at dry season flow to be about 620 mg/l (existing plant) and 700 mg/l (including Phase II) for BOD_5 and 800 mg/l and 910 mg/l respectively for SS (Gauff... 1976).

The main units of the treatment plant are:

1. screens
2. oxidation ditch equipped with TNO cage rotors
3. two settling tanks with mechanical scrapers
4. two screw pumping sets
5. sludge thickener
6. eight sludge drying beds.

The effluent is discharged into a natural pond.

Figures 12 and 13 show general layouts of the treatment plant and Table 7 shows the main components.

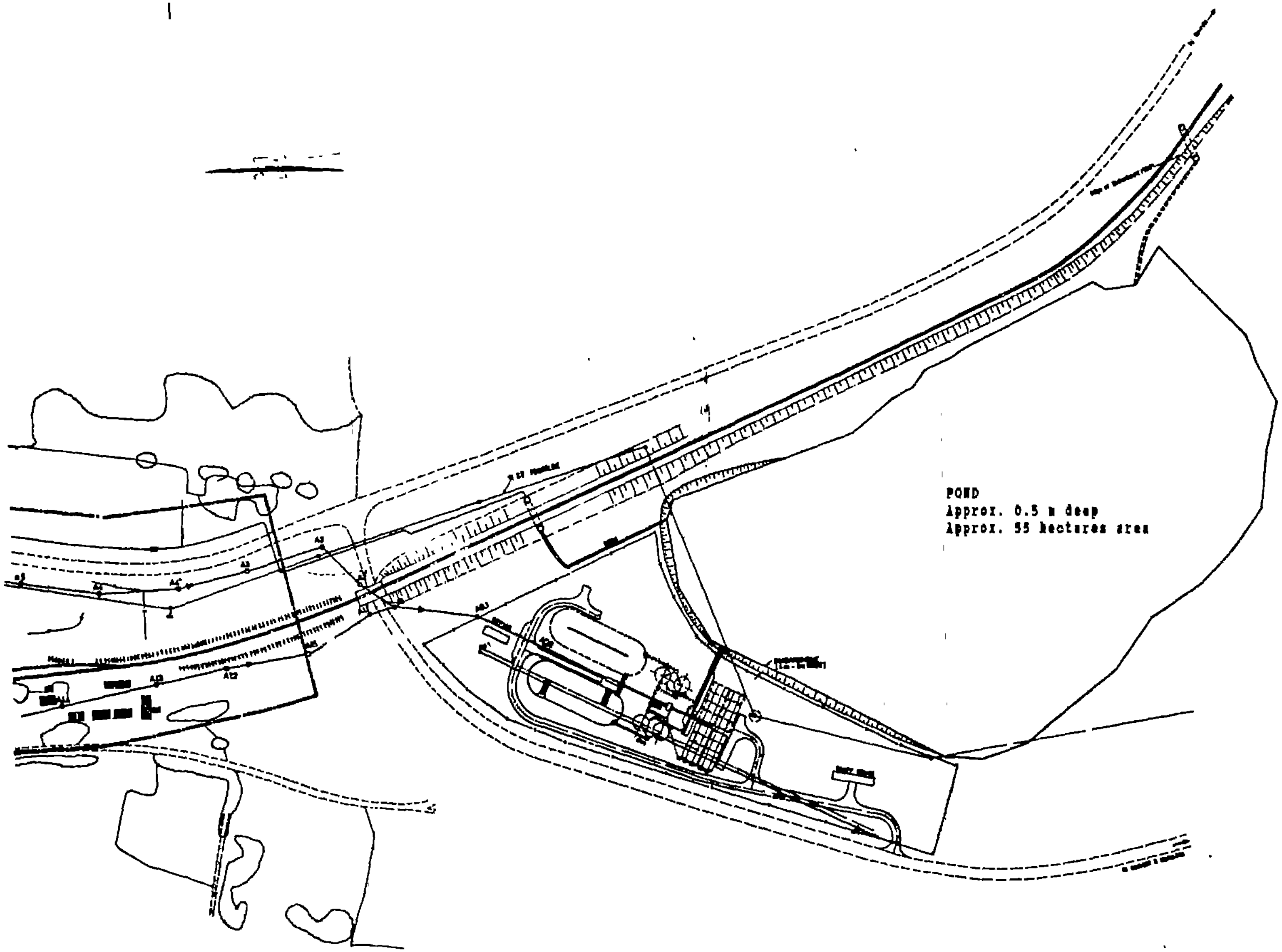

Figure 12. General layout of the treatment plant, including natural pond (Gauff... 1978).

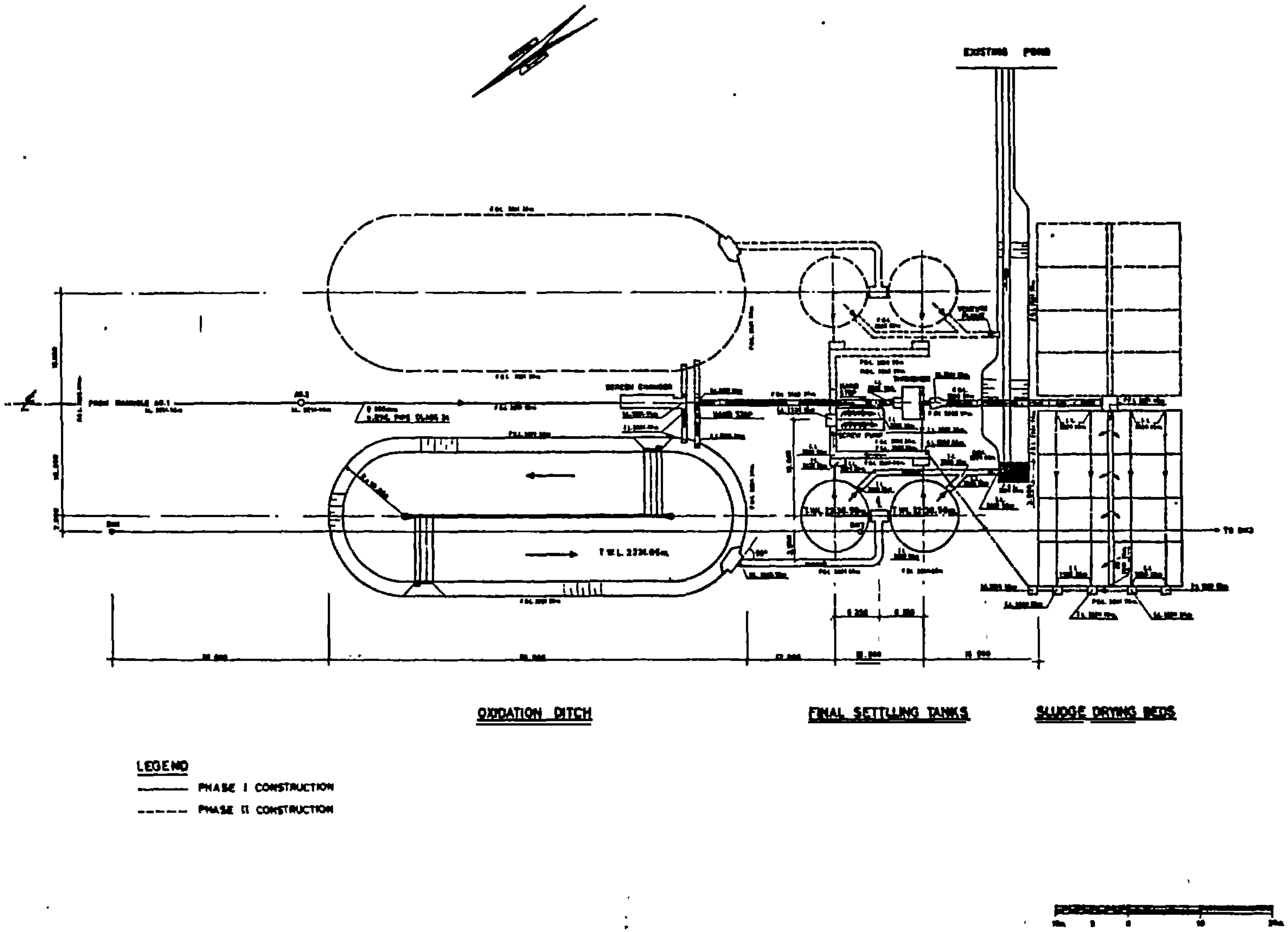

Figure 13. General layout of the treatment plant, excluding natural pond (Gauff... 1978).

Table 7. Main stages of the treatment plant (Gauff... 1978).

Stage	Description
Screens	3 sets - 40 x 13 mm steel bars 80 mm apart - 40 x 10 mm steel bars 40 mm apart - 40 x 6 mm steel bars 25 mm apart
Oxidation ditch	1 aeration tank 56.75 x 20.20 x 1.80 m 2 nos, 7 m TNO cage rotors driven by 11 kW motors. Aeration capacity 1.34 kg/m/h at 75 mm immersion to 3.73 kg/m/h at 200 mm immersion.
Settling tanks	2 nos, 7.75 m internal diameter 1.6 m side water depth equipped with mechanical scrapers.
Screw pumps	2 nos, 400 mm diameter screws with 2.2 kW drive units.
Sludge drying beds	8 nos, each 9.2 x 5.5 m

Different parts of the treatment plant are shown in Figures 14 to 19.

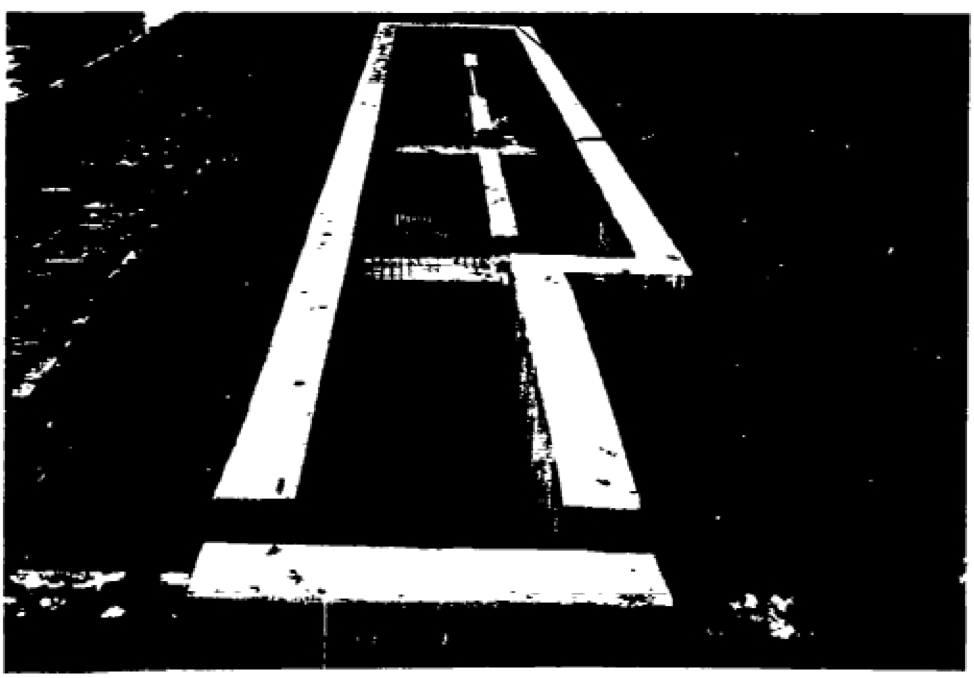

Figure 14. Screens.

Figure 15. Oxidation ditch with TNO cage rotors.

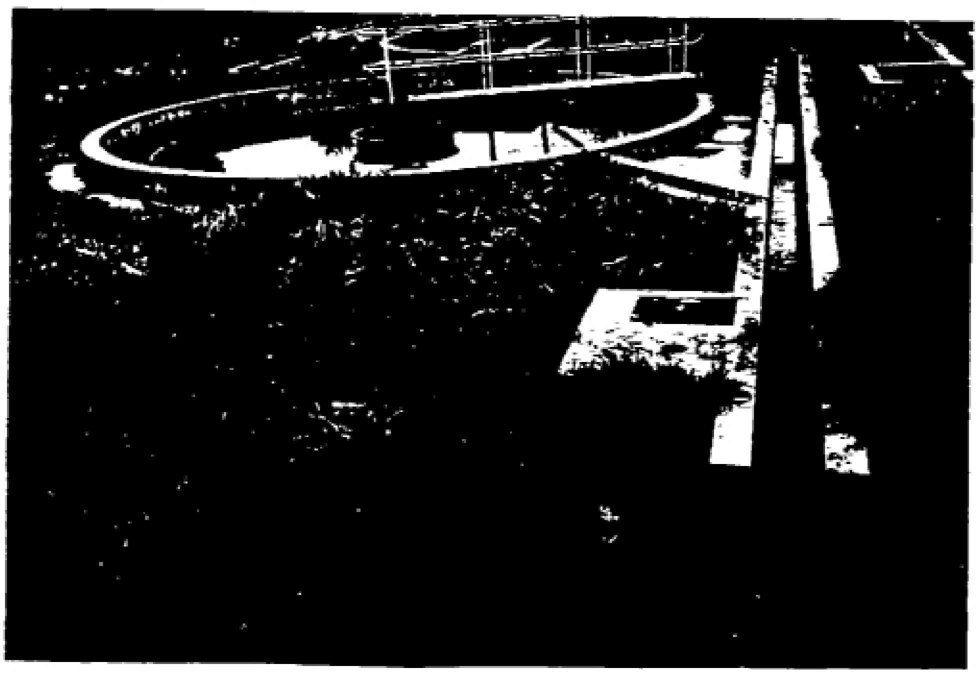

Figure 16. Settling tanks and the outlet channel.

Figure 17. Effluent drainage ditch. The natural pond is in the background.

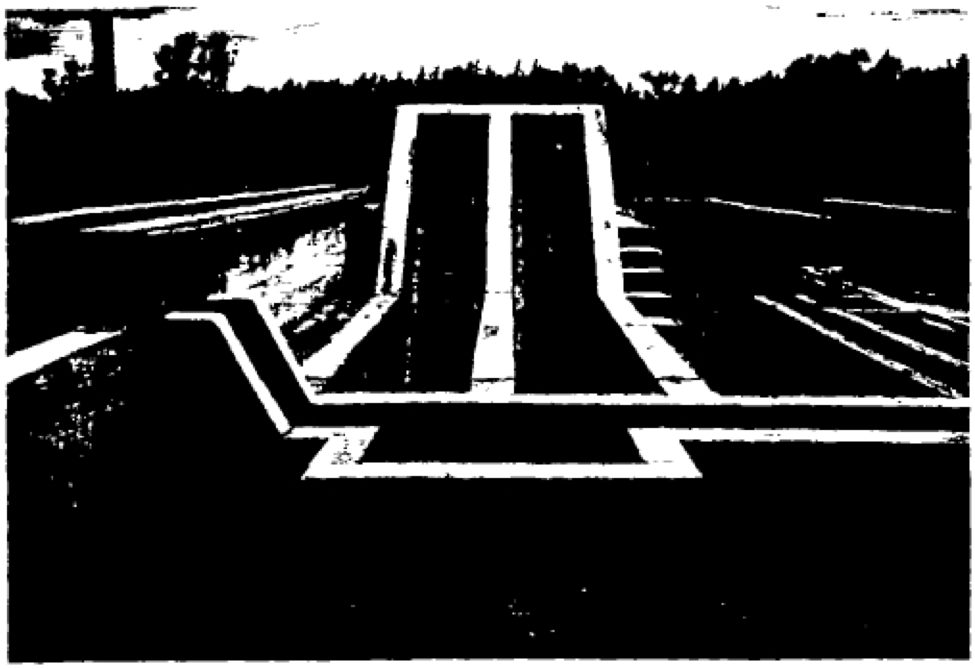

Figure 18. Screw pumps.

Figure 19. Sludge drying beds.

3.2 Investigation procedures

Treatment plant influent, mixed liquor in the ditch and effluent were sampled twice a week between 7.11.1989 - 11.1.1990. pH and temperature at the inlet, in the oxidation ditch and at the outlet were measured (WTW OXI 96 meter) twice a day at 10.00 a.m. and 3.00 p.m between 5. - 11.1.1990. DO and temperature in the oxidation ditch were measured (WTW meter) at ten different points three times a day at 11.00 a.m., 1.00 p.m. and 3.00 p.m. at a distance of 1.5 m from the perimeter of the ditch and 1.5 m deep. This was done between 5. - 11.1.1990. 24 h flow measurements were taken using a 60° V-notch weir between 19. - 30.12.1989 and 2. - 4.1.1990.

Types of samples collected are shown in Table 8.

Table 8. Types of samples.

Sampling point	Type of sample
Inlet	G
Oxidation ditch	G
Outlet	C, G

G = grab sample
C = composite sample collected using a 24 h automatic effluent sampler type EPS 1011 (Epic Products Ltd, UK). Samples were not flow-weighted.

All grab samples for bacteriological analysis were collected using sterile 300 ml bottles. All the other samples were collected using clean plastic containers. An ice box was used to transport the samples to the University of Nairobi for analysis.

The visibility depths in the settling tanks were measured using the Secchi disc.

Field measurements and laboratory analyses, excluding flow measurements, are shown in the appendices.

4 RESULTS AND DISCUSSION

4.1 Sampling and laboratory analyses

The following factors affected the sampling programme:

1. available equipment

2. the capability of the laboratory to handle many samples at a time

3. distance between the treatment plant and the laboratory

4. the five-day working week.

Taking grab and composite samples at the same points would have been the most satisfactory approach. There was one automatic sampler available and it could be used at only one point at a time. And even where the automatic sampler was used, the composite samples could not be flow-proportioned due to possibilities of errors arising out of manual flow-measurements if carried out for 24 hours. Taking measurements at night in Limuru is not safe because of the prevalence of thuggery.

To determine the treatment efficiency and economic operation in wastewater treatment plants, examination of physical facilities and unit operations is necessary. This requires analysing all influent and effluent flow streams (Hammer 1977). Loading could not be calculated for each unit operation due to lack of flow measuring facilities and the need for many samples. The following parameters, although important were omitted due to lack of facilities:

1. sludge age
2. total N
3. NH_4 - N
4. total P
5. flow velocity in the oxidation ditch.

4.2 Influent and effluent characteristics

4.2.1 Wastewater flow measurements

Ball (1970) points out that effective treatment requires information regarding rate of flow. Physical facilities for flow-measuring and sampling at various points are essential. Lack of adequate flow-measuring facilities makes the study of unit operations difficult (Hammer 1977).

Influent flow-measurements were difficult as the weir installed for the purposes of the study could not cope with the flow at certain times of the day when the wastewater flooded the inlet channel and some of it bypassed the weir.

4.2.2 Influent and effluent BOD_5

Table 9 shows the influent and effluent BOD_5. Figure 20 shows the same values graphically.

Table 9. Influent and effluent BOD_5.

Date	Influent BOD_5 mg/l	Effluent BOD_5 mg/l	Reduction %
08.11.1989	470	125	73
09.11.1989	560	146	74
15.11.1989	1320	120	91
16.11.1989	880	110	88
22.11.1989	2500	160	94
23.11.1989	790	150	81
29.11.1989	784	150	81
30.11.1989	2200	180	92
06.12.1989	760	120	84
07.12.1989	1015	175	83
13.12.1989	2775	180	94
14.12.1989	1079	126	88
20.12.1989	830	705	15
21.12.1989	1050	920	12
03.01.1990	126	110	13
04.01.1990	1050	260	75
10.01.1990	800	125	84
11.01.1990	890	260	71
Range	126 - 2775	110 - 920	12 - 94
Average	1100	230	72

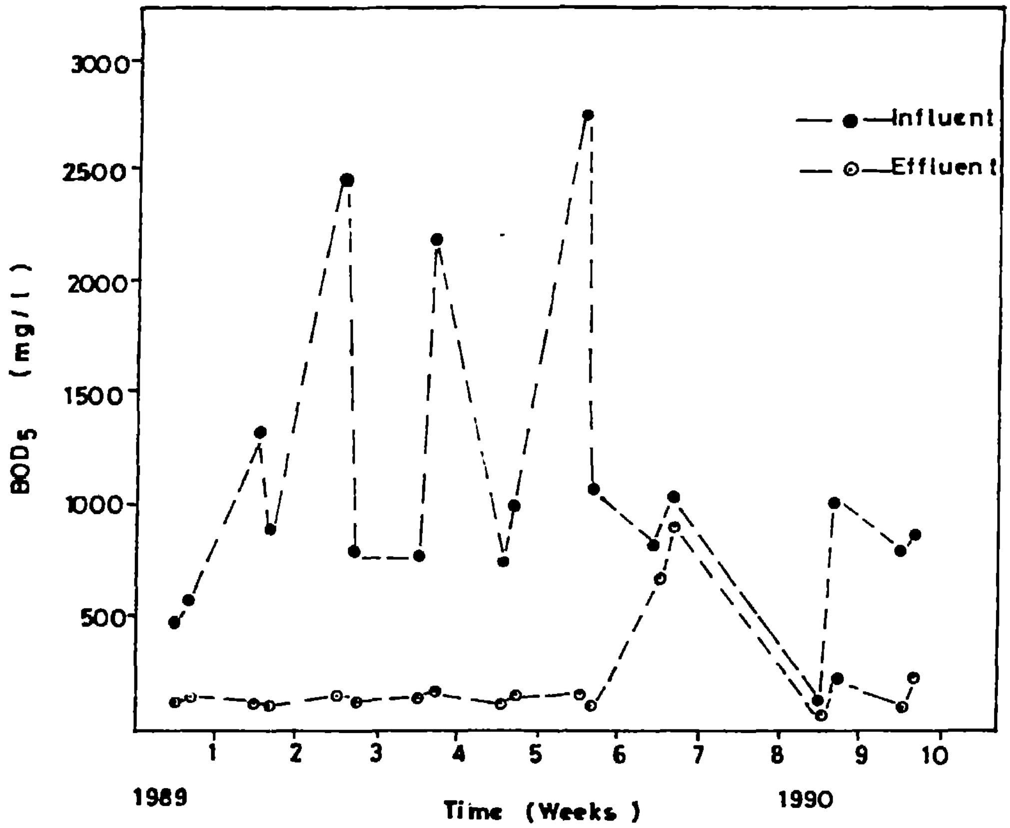

Figure 20. Influent and effluent BOD_5.

The range between the lowest and highest value is very wide (126 - 2 775 mg/l). The low value could be attributed to dilution caused by storm water.

Wastewater strength is most often judged by its BOD_5 or COD (Table 10).

Table 10. Sewage strength in terms of BOD_5 and COD (Mara 1978).

Strength	BOD_5 mg/l	COD mg/l
Weak	< 200	< 400
Medium	350	700
Strong	500	1000
Very strong	> 750	1500

The influent BOD_5 values show that the wastewater entering the plant is very strong. This is rather unexpected as it is mostly domestic, with small contributions from the slaughterhouse and the PVC pipe factory. The high BOD_5 values may partly be explained by water consumption. It is also possible that values represent those on the higher side since they were obtained from grab samples taken at about the same time every day, a time when concentrations may be very high. The daily per capita BOD_5 contributions may also offer an explanation.

None of the effluent BOD_5 values met the 20 mg/l limit set by the Ministry of Water Development Pollution Control Division. The lowest value is about 6 times the limit while the highest value is 46 times and this obviously means very low quality effluent.

An indication of the efficiency of the treatment plant has been obtained by calculating the percentage reductions. Although some samples give reductions over 90 %, generally the efficiency of the plant is poor compared with the performance of plants reported elsewhere (Heide 1982, Mandt and Bell 1982). It should be noted that grab samples were analysed for the influent BOD_5 while 24 h composite samples were used for analysing the effluent BOD_5. The same limitation applies to all the other parameters concerning the influent and effluent except fecal coliforms.

4.2.3 Influent and effluent COD

Table 11 and Figure 21 show COD values.

Table 11. Influent and effluent COD values.

Date	Influent COD mg/l	Effluent COD mg/l	Reduction %
08.11.1989	1280	160	88
09.11.1989	1128	296	74
15.11.1989	3120	512	84
16.11.1989	2320	256	89
22.11.1989	4480	440	90
23.11.1989	1040	320	69
29.11.1989	1060	260	75
30.11.1989	3800	320	92
06.12.1989	1080	75	93
07.12.1989	1888	240	87
13.12.1989	2816	256	91
14.12.1989	1552	266	83
20.12.1989	1120	208	81
21.12.1989	1318	310	76
03.01.1990	381	240	37
04.01.1990	2160	640	70
10.01.1990	1060	176	83
11.01.1990	1090	368	66
Range	381 - 4480	75 - 640	37 - 93
Average	1816	297	79

The range between the lowest influent COD value (381 mg/l) and the highest (4 480 mg/l) is very wide. The lowest COD value was obtained on the same day the lowest BOD value was obtained and this could be explained why storm water diluting the wastewater. In terms of COD, the average 1 816 mg/l indicates that the wastewater is very strong. The wide variations in the values can be seen in Figure 21.

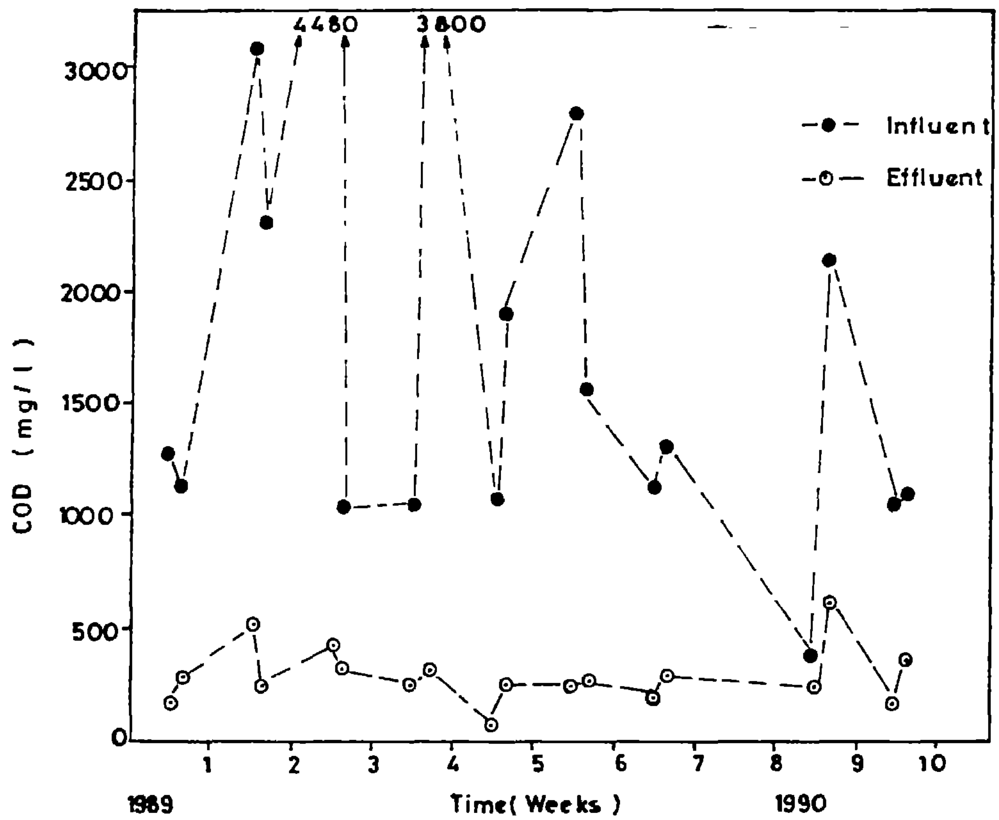

Figure 21. COD values.

The effluent COD values ranging between 75 - 640 mg/l with a mean of 297 mg/l indicate low quality effluent if compared with the 120 - 140 mg/l limit used in the Federal Republic of Germany (Kollatsch and Gowasch 1982). Only one sample meets these discharge limits.

4.2.4 Influent and effluent SS

SS variations are shown in Table 12.

Table 12. Influent and effluent SS values.

Date	Influent SS mg/l	Effluent SS mg/l	Reduction %
08.11.1989	544	68	88
09.11.1989	488	204	58
15.11.1989	1310	352	73
16.11.1989	730	60	92
22.11.1989	1360	120	91
23.11.1989	532	304	43
29.11.1989	1330	90	93
30.11.1989	1790	150	92
06.12.1989	884	66	93
07.12.1989	1590	68	96
13.12.1989	1340	66	95
14.12.1989	660	108	84
20.12.1989	890	130	85
21.12.1989	904	210	77
03.01.1990	175	360	- 51 *)
04.01.1990	1410	230	84
10.01.1990	496	74	85
11.01.1990	1300	510	61
Range	175 - 1790	60 - 510	43 - 96
Average	985	176	74

*) Concentrations in the effluent higher than influent.

There are wide variations in the influent SS concentrations (175 - 1 790 mg/l). The mean of 985 mg/l is higher than the design value of 805 mg/l. This should not be interpreted to mean temporary overloading as these values were obtained from grab samples.

The SS concentrations in the effluent fell in the range 60 - 510 mg/l, with an average of 176 mg/l. None of the samples satisfied the 30 mg/l limit.

The reductions in SS concentrations were in the range 43 - 96 % with an average of 74 %. In one case the effluent SS value was higher than the corresponding influent value. This sample was taken after heavy rains and the storm water diluted the influent while washing out accumulated solids from the system.

4.2.5 Fecal coliform organisms

Fecal coliform counts are shown in Table 13.

Table 13. Fecal coliform counts.

Date	Influent counts MPN/100 ml	Effluent counts MPN/100 ml	Reduction %
08.11.1989	360 x 10^3	128 x 10^3	64
09.11.1989	340 x 10^3	130 x 10^3	62
15.11.1989	134 x 10^3	119 x 10^3	11
16.11.1989	170 x 10^3	129 x 10^3	24
22.11.1989	330 x 10^3	123 x 10^3	63
23.11.1989	337 x 10^3	122 x 10^3	64
29.11.1989	340 x 10^3	180 x 10^3	47
30.11.1989	342 x 10^3	190 x 10^3	44
06.12.1989	320 x 10^3	175 x 10^3	45
07.12.1989	338 x 10^3	200 x 10^3	41
13.12.1989	330 x 10^3	195 x 10^3	41
14.12.1989	300 x 10^3	198 x 10^3	31
20.12.1989	310 x 10^3	196 x 10^3	37
21.12.1989	240 x 10^3	30 x 10^3	88
03.01.1990	108 x 10^3	96 x 10^3	11
04.01.1990	103 x 10^3	100 x 10^3	2.9
10.01.1990	106 x 10^3	105 x 10^3	0.9
11.01.1990	245 x 10^3	196 x 10^3	20
Range	103 - 360 x 10^3	30 - 200 x 10^3	0.9 - 88
Average	264 x 10^3	145 x 10^3	40

As expected, the reductions in most cases were not satisfactory as can be seen from the wide range (0.9 -88 %) and low average (40 %). This confirms Mara's (1978) recommendation that provisions should always be made for the bacteriological improvement of the effluent in oxidation ditch plants. To ensure that effluents produced in treatment plants do not pollute the receiving water courses, Mara (1978) proposes < 5 000 cells/100 ml as the minimum effluent standard.

The effluent values obtained in this study are lower than values considered typical for oxidation ditch plant by Goronszy (1979). In fact, effluent values reported by Goronszy (1979) are very close to those of the influent in this study.

4.2.6 Influent and effluent pH

The pH values for the influent and effluent are shown in Table 14.

Table 14. Influent and effluent pH values.

Sample	pH range	pH mean
Influent	7.4 - 8.0	7.7
Effluent	7.7 - 8.1	7.9

For most wastewater treatment processes the pH range for growth falls somewhere between pH 4 - 9. The optimum pH for growth lies between pH 6.5 - 7.5. Biological wastewater treatment processes are seldom operated at optimum conditions and experience has shown that extended aeration activated sludge plants can successfully be operated when the pH range is pH 9 - 10.5. But they are very vulnerable to pH levels below 6 (Benefield and Randall 1985). It can, therefore, be concluded that the raw wastewater is treatable in the oxidation ditch in respect of pH.

The effluent pH values lie within the acceptable range of pH 6 - 9.

4.3 Activated sludge characteristics

4.3.1 DO concentrations

The measuring points for DO concentrations are shown in Figure 22 and the frequency of the stated concentrations in Table 15.

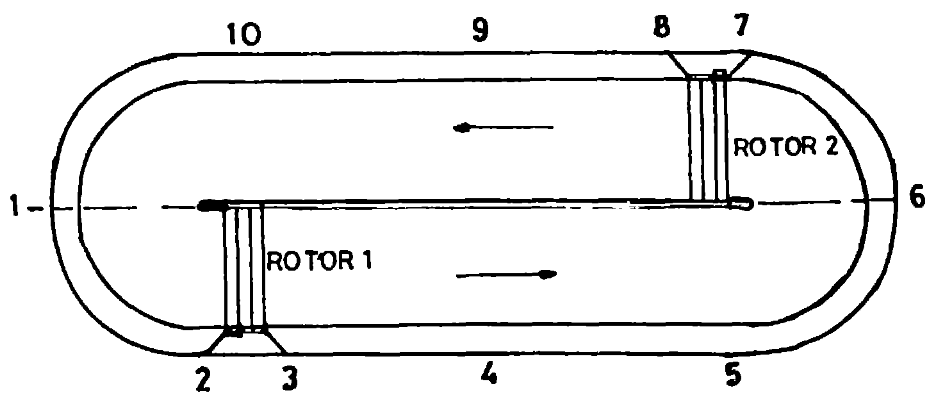

Figure 22. DO measuring points.

Table 15. Frequency of stated DO concentrations.

DO concentration (actual values) mg/l	Measuring points									
	1	2	3	4	5	6	7	8	9	10
	Number of observed consentration									
0		1	1	1	2	1	3	3	2	3
0.1	8	8	6	8	9	9	8	9	9	9
0.2	10	11	12	11	10	10	10	9	10	8
0.3	3	1	2	1		1				1

The DO concentration was either 0.1 or 0.2 mg/l most of the time as shown by the frequency of the stated concentrations in Table 15. There was no detectable DO at least once for every point except point 1. DO level 0.3 mg/l was recorded at least once at six points.

Examination of the results does not show any trend in the DO concentration around the ditch. This does not agree with Rittmann and Langeland (1985) who found that the lowest concentration occurred at the furthest point from the aerator while the highest concentration was found directly after the aerator.

Both rotors were operating when measurements were taken on 05.01.1990 at 1.00 p.m. and the DO concentration was 0.1 mg/l at all the points. The uniform values obtained around the ditch suggest an improvement in the mixing of ditch contents when both rotors are working. However, the low DO concentration was rather unexpected as two similar rotors operating at the same time would be expected to provide more oxygen than one rotor.

Grube and Murphy (1969) plotted oxygen profiles at depths of 0.15 m and 0.60 m and they found the concentration gradient steep, indicating that the oxygen demand of the mixed liquor was not constant throughout the entire volume of the ditch. Similar results were obtained by Rittmann and Langeland (1985). The good mixing and the relatively shallow ditch in this study make the results obtained a good indication of the DO concentrations although measurements were taken at one depth only.

The DO concentrations in the ditch can be regarded very low. The manufacturers of the aeration equipment recommend that oxygen levels should be maintained in the range 1 - 2 mg/l. Arceivala (1986) points out that DO levels less than 0.5 mg/l must be avoided as they tend to favour the growth of filamentous organisms and increase the SVI.

According to Simpson (1964), inadequate aeration will produce anaerobic conditions in the aeration and settling tanks, leading to a poor quality effluent. On the other hand, excessive aeration produces "pin-point" flocs with poor settling properties (Simpson 1964).

The anoxic conditions in the oxidation ditch throughout the study period seem to explain partly the poor effluent from the plant.

4.3.2 MLSS, MLVSS, SVI, and pH

Table 16 shows values of MLSS, MLVSS and SVI. Variations in MLSS and SVI are also shown in Figures 23 and 24.

Table 16. MLSS, MLVSS and SVI values.

Date	MLSS mg/l	MLVSS mg/l	Settled sludge volume ml/l	MLVSS/ MLSS ratio	SVI ml/g
08.11.1989	9600	7950		0.83	
09.11.1989	2070	1250		0.60	
15.11.1989	1220	390		0.32	
16.11.1989	1300	245		0.19	
22.11.1989	12170	7590		0.62	
23.11.1989	12140	6940		0.57	
29.11.1989	2860	1750		0.61	
30.11.1989	3020	1900		0.63	
06.12.1989	10840	8960		0.83	
07.12.1989	8650	6860		0.79	
13.12.1989	10820	8420		0.78	
14.12.1989	10560	8140	460	0.77	44
20.12.1989	9080	7160	310	0.79	34
21.12.1989	10100	7040	290	0.70	29
03.01.1990	890	790	400	0.89	449
04.01.1990	4180	3220	160	0.77	38
10.01.1990	5650	4560	350	0.81	62
11.01.1990	6820	4980	300	0.73	44
Range	890-12170	245-8960	160-460	0.19-0.89	29-449
Average	6780	4900	320	0.68	100 42 *)

\- measurements not taken
*) excluding the atypical value of 449

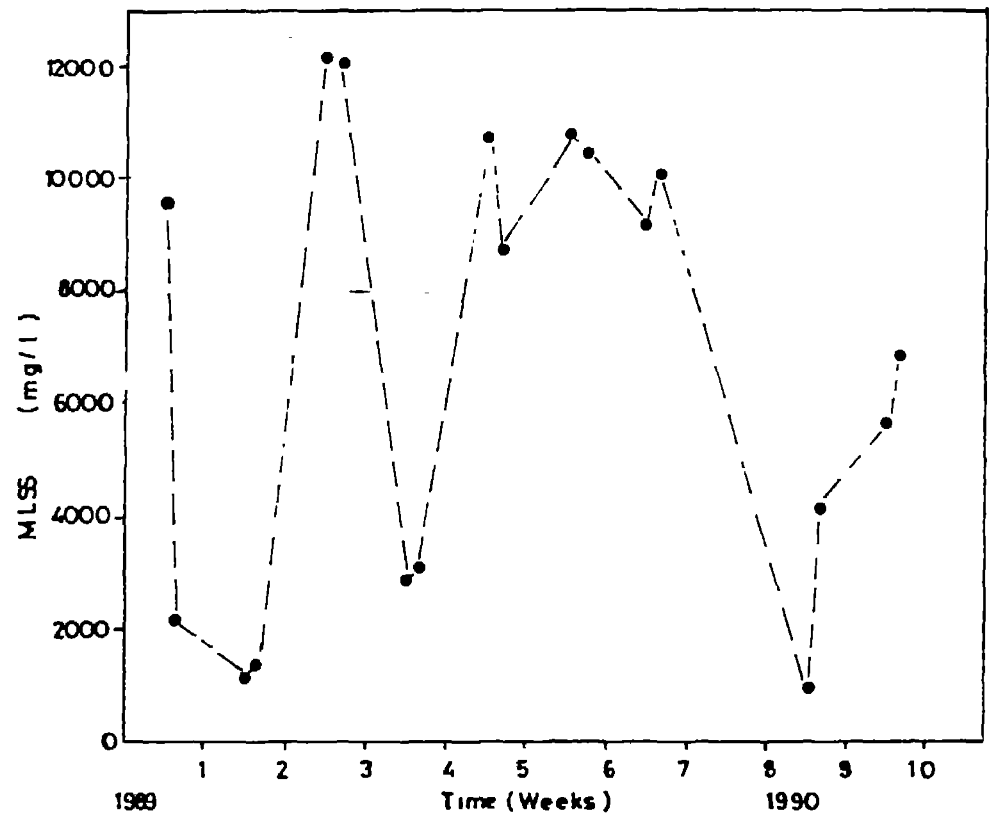

Figure 23. MLSS variations.

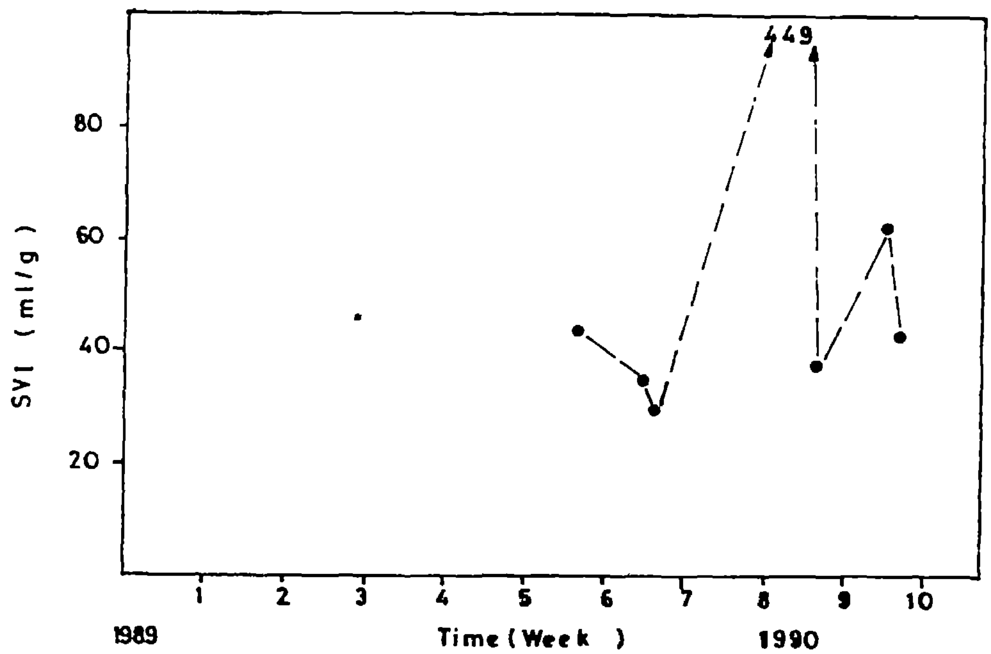

Figure 24. SVI variations.

Examination of the MLSS values show wide fluctuations. These wide fluctuations are an indication of poor process control.

The lowest value (890 mg/l) was recorded after heavy rains. This low value indicates an operational problem. The practice after heavy rains is to adjust the effluent weir to the lowest position and at times even switch the rotors off. Apart from allowing the wastewater to flow through the treatment plant with practically no treatment, there is a danger of the biomass being washed out of the process.

Recommended ranges for the MLSS concentrations have been reported by several authors, for example 2 000 - 5 000 mg/l (Barnes and Wilson 1978, cited by Gemmell and Herbert 1985) and 2 000 - 6 000 mg/l (Water Pollution Control Federation Manual of Practice No. 8, 1982, cited by Gemmell and Herbert 1985) and 3 000 - 6 000 mg/l (Metcalf and Eddy 1979). The Water Pollution Control Federation Manual of Practice recommends the widest range and using this as a basis of analysis, 10 samples had values significantly exceeding the upper limit of 6 000 mg/l, 5 samples fell within the recommended range and 3 were below the lower limit of 2 000 mg/l. Very low MLSS concentrations will lead to partial treatment of the waste, and according to Pitman (1978), the respiration demands of maintaining the micro-organisms at very high concentrations could exceed the aeration capacity of the rotors.

Operation and maintenance instructions recommend a value of 3 500 mg/l for the MLSS. The wide deviations from the recommended value are an additional indicator of the process being out of control.

Examination of the MLVSS/MLSS ratios shows 13 samples falling within the design values (0.5 - 0.8) proposed by Arceivala (1986). Two samples fall below the lower limit and three samples fall above the upper limit, with the highest ratio being about 11 % higher than the upper limit. However, the average falls within the recommended range.

The MLVSS/MLSS ratio is of significance in the USA where MLVSS is preferred over MLSS with the ratio 0.7 being used for design purposes (Scott and Smith 1980).

A properly settling sludge has the SVI varying between 40 - 100 (Tebbutt 1983). The findings indicate the sludge with good settling properties and this agrees with obervations on the mixed liquor in the sampling bottles immediately after sampling.

The pH values of the mixed liquor in the oxidation ditch fell in the range pH 7.7 - 8.0 with a mean of pH 7.8, which is within the acceptable range for biological wastewater treatment processes.

4.3.3 Visibility depths in the settling tanks

The visibility depths in the settling tanks are shown in Table 17.

Table 17. Visibility depths in the settling tanks.

Date	Tank 1 depth cm	Tank 2 depth cm
05.01.1990	25	25
06.01.1990	20	25
07.01.1990	25	25
08.01.1990	25	20
09.01.1990	20	25
10.01.1990	20	25
11.01.1990	25	20
Average	23	24

Examination of plant records show that depths of up to 125 cm have been observed. The values in Table 17 indicate a progressive deterioration in the performance of settling tanks over a period of time. This poor performance could be attributed to the worn out rubbers on the scrapers. The rubbers have been available for more than four years but have not been fixed due to lack of desludging equipment.

4.4 Factors limiting treatment plant performance

Limuru wastewater treatment plant is producing a very poor quality effluent. Factors limiting treatment plant efficiency are discussed below.

4.4.1 Design aspects

Although provisions were made for storm water in the design, it was a serious problem. According to Gray et al (1980), there are two problems associated with the periodic surging. One problem is the "washout" of the suspended growth system during the high flow periods. The other problem is the dilution of raw wastewater with the result that the biological process loadings are much less than design intent, and a healthy biomass cannot be sustained.

Influent characteristics show that the wastewater is stronger than assumed in the design in terms of BOD_5 and SS. The high BOD_5 and SS concentrations offer a possible explanation for the poor performance.

Process control is another problem. There are no storm water bypass facilities, and during the rains the oxidation ditch effluent weir is adjusted to its lowest position and the rotors sometimes switched off to maintain recommended immersion depths. Apart from discharging the wastewater virtually untreated, control of the process is difficult at such times.

DO levels did not exceed 0.3 mg/l during the study. The low DO levels could be a contributory factor limiting the wastewater treatment plant performance.

Inadequate capacity of the sludge drying beds is said to be a constraint, especially during the wet season. This could well be true but observations indicate that no efforts have been made to optimize their use.

4.4.2 Operational factors

One of the problems facing the treatment plant staff is the lack of laboratory equipment. No tests are carried out for process control. A small laboratory stopped functioning, when all the equipment were transferred to Nairobi.

Although almost all operators are trained, they do not seem to apply the right treatment concepts most of the time. For example, sludge is returned to the oxidation ditch from the sludge thickener when the sludge drying beds are full.

4.4.3 Maintenance factors

Inadequate attention is paid to emergency and routine maintenance. At one time or another, the treatment process has been disrupted by the following items:

1. main switchboard
2. screw pumps
3. rotors.

The filter media in the sludge drying beds is inadequate and resanding is necessary. The rubbers on the settling tank scrapers need replacement and the time taken to repair or replace damaged items is rather too long.

4.4.4 Management

The workload does not seem to be a problem. There are eight workers. All workers have job descriptions although there have been occasions when they have been instructed to undertake duties not conforming to the job descriptions. Although this can be a source of discontent, it does not seem to have adversely affected the treatment plant performance.

Co-ordination and supervision by the District Water Office and Water Development Head Office has been affected by the workload in those offices and the lack of facilities. It seems that this may explain why the workers are underutilized.

Budgeting and other finance related matters are not handled at the treatment plant and are therefore beyond the control of the treatment plant staff. But budgetary ceilings set by the Treasury are a big constraint. It is likely that administrative deficiencies resulting from budgetary ceilings are partially or wholly responsible for the shortcomings already identified.

5 CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

1) Oxidation ditch plants are capable of consistently producing high quality effluents which has made the technology firmly established and accepted as a treatment method. Good performance is not, however, achieved at Limuru as explained below:

   a) The BOD_5 removal efficiencies were in the range 12 - 94 % with an average of 72 %. None of the effluent samples met the 20 mg/l BOD_5 limit. The lowest value was about 6 times the limit and the highest value 46 times.

   b) The COD removal efficiencies were in the range 37 - 93 % with an average of 79 %. The effluent COD values were in the range 75 - 640 mg/l with a mean of 297 mg/l and only one sample met the set limits.

   c) The SS concentrations in the effluent were in the range 60 - 510 mg/l with an average of 176 mg/l. None of the samples met the 30 mg/l limit. The removal efficiencies were in the range 43 - 96 % with an average of 74 %.

   d) The visibility depths in the settling tanks were 23 cm and 24 cm on the average. Depths of 125 cm have been recorded and the shallow depths observed in the study are an indication of the progressive deterioration of the performance of settling tanks.

2) The poor performance of the Limuru treatment plant can be attributed to inadequacies in design, operation, maintenance and management. Some of the inadequacies are:

   a) Wide variations were observed in the influent BOD_5 and COD while variations are to be expected, what is of concern is the fact that most of the values observed indicate a very strong sewage in terms of BOD and COD respectively. The BOD_5 design value was 614 mg/l.

   b) The influent SS concentrations were in the range 175 - 1 790 mg/l with a mean of 985 mg/l. 11 out of the 18 the 18 samples exceeded the design value of 805 mg/l.

   c) Observations indicated that the treatment plant is not capable of handling large flows of storm water.

d) The DO concentration in the ditch was either 0.1 or 0.2 mg/l most of the time. Concentrations 0 and 0.3 mg/l were also observed. The concentrations can be regarded very low bearing in mind the equipment manufacturer's recommended range 1 - 2 mg/l. Inadequate aeration favours the growth of filamentous organisms and can produce anaerobic conditions in the aeration and settling tanks. The result is poor effluent quality.

e) The poor process control is evident from the wide range of the MLSS values (890 - 12 170 mg/l, average 6 780 mg/l). 12 samples out of 18 significantly exceeded the equipment manufacturer's recommended value of 3 500 mg/l, the mean being about 2 times this value. Too high levels of MLSS could exceed the aeration capacity of the rotors while levels which are too low will result in partial treatment of the sewage.

f) No laboratory analyses can be carried out to monitor process performance and take remedial action when necessary due to lack of laboratory facilities.

g) The majority of the operators are trained but the right treatment concepts do not seem to be applied most of the time.

h) Inadequate attention has been given to maintenance with the result that breakdowns take long to be attended to. The period between requisition and supply of spare parts, tools and materials is long.

i) Co-ordination and supervision by the District Water Office and Water Development Headquarters is hampered by the workload in those offices and the lack of facilities.

j) Budgetary ceilings set by the Treasury are a big constraint.

3) a) The SVI was in the range 29 - 62 with an average of 42 and this indicates sludge with good settling properties.

b) The pH ranges for the influent (7.4 - 8.0) and oxidation ditch mixed liquor (7.7 - 8.0) were within the range in which biological processes can take place.

c) The pH of the effluent fell in the range 7.7 - 8.1 and the values were well within the discharge requirements.

5.2 Recommendations

1) The effluent BOD_5 and SS values seem to indicate overloading. Storm water needs attention. The sources of sewage and the entry points of the storm water have to be studied to check the conclusions made in this study and to check design assumptions.

2) The rotors were incapable of maintaining desirable DO levels and there is need to check whether this has been caused by overloading or other factors not identified in this study.

3) Although most of the operators are trained, they do not seem to be applying the knowledge gained during training in the operation of the treatment process. It will be necessary to organize on-the-job training to be conducted by instructors who understand the oxidation ditch technology.

   Follow-ups will be necessary to ensure conformity with acceptable operation and maintenance procedures, compliance with discharge requirements and offer advice where necessary. A scrutinity of the curriculum may be desirable.

4) Operation and maintenance organizational structure and procedures need to be reviewed at the treatment plant, the District Water Office and the Ministry of Water Head Office. The review will facilitate identification of deficiencies responsible for delays in decision making.

5) Laboratory facilities and flow-measuring instruments are essential for process monitoring and these should be provided. But the provision of these facilities will not itself improve the performance of the plant.

   Chemicals will continue to be a problem for a long time to come and there is need to identify parameters that can adequately give an idea of effluent quality but requiring minimal use of chemicals. Effluent pH and SS can be analysed easily and cheaply. Measuring the visibility depths in the settling tanks is equally easy. A simple settling test could be used to decide when to return or remove sludge. These tests could be done as frequently as possible and supplemented by BOD_5 determinations once in a week or even less frequently depending on the consistencies of the parameters analysed frequently.

6) A lot of things at the treatment plant could be done using common sense. Before the operators can be asked to use common sense, there is need to undertake an educational campaign to change the operators' attitude towards their work. There are factors that led to the development of this negative attitude and identifying and addressing each of them may turn out to be challenging but rewarding if successfully accomplished.

7) In the long run, it will be beneficial to make the treatment plant self-supporting by setting appropriate effluent discharge tariffs and ensuring that all the money collected is used for sewage-related activities only. This is a policy issue which will require sound advice from engineers regarding the merits and demerits of the existing and proposed tariff systems.

8) Optimization of the treatment plant performance is essential. This will require intensive studies spread over a long period of time and incorporating as many parameters as possible. This could be undertaken at the same time with the on-the-job training. To successfully undertake this exercise, it will be necessary to have adequate laboratory and backup facilities.

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APPENDIX 1

Summary of BOD_5, SS, COD and fecal coliform count results.

Date	Sample	BOD_5	SS	COD	Faecal coliform counts MPN/100 ml
8.11.1989	Influent	470	544	1 280	360×10^3
	Effluent	125	68	160	128×10^3
9.11.1989	Influent	560	488	1 128	340×10^3
	Effluent	146	204	296	130×10^3
15.11.1989	Influent	1 320	1 310	3 120	134×10^3
	Effluent	120	352	512	119×10^3
16.11.1989	Influent	880	730	2 320	170×10^3
	Effleunt	110	60	256	129×10^3
22.11.1989	Influent	2 500	1 360	4 480	330×10^3
	Effluent	160	120	440	123×10^3
23.11.1989	Influent	790	532	1 040	337×10^3
	Effluent	150	304	320	122×10^3
29.11.1989	Influent	784	1 330	1 060	340×10^3
	Effluent	150	90	260	180×10^3
30.11.1989	Influent	2 200	1 790	3 800	342×10^3
	Effluent	180	150	320	190×10^3
6.12.1989	Influent	760	884	1 080	320×10^3
	Effluent	120	66	75	175×10^3
7.12.1989	Influent	1 015	1 590	1 888	338×10^3
	Effluent	175	68	240	200×10^3
13.12.1989	Influent	2 775	1 340	2 816	330×10^3
	Effluent	180	66	256	195×10^3
14.12.1989	Influent	1 079	660	1 552	300×10^3
	Effluent	126	108	266	198×10^3
20.12.1989	Influent	830	890	1 120	310×10^3
	Effluent	705	130	208	196×10^3
21.12.1989	Influent	1 050	904	1 318	240×10^3
	Effluent	920	210	310	30×10^3
3. 1.1990	Influent	126	175	381	108×10^3
	Effluent	110	360	240	96×10^3
4. 1.1990	Influent	1 050	1 410	2 160	103×10^3
	Effluent	260	230	640	100×10^3
10. 1.1990	Influent	800	496	1 060	106×10^3
	Effluent	125	74	176	105×10^3
11. 1.1990	Influent	890	1 300	1 090	245×10^3
	Effluent	260	510	368	196×10^3

APPENDIX 2

pH and temperature data.

Date	Time	Point	pH	Temperature °C
5.1.1990	10.00 a.m.	Inlet	*	*
		Oxidation ditch	*	*
		Outlet	*	*
	3.00 p.m.	Inlet	7.4	18.8
		Oxidation ditch	7.8	20.2
		Outlet	7.7	21.3
6.1.1990	10.00 a.m.	Inlet	7.7	18.3
		Oxidation ditch	7.8	19.2
		Outlet	7.7	19.1
	3.00 p.m.	Inlet	7.7	19.0
		Oxidation ditch	7.7	20.5
		Outlet	7.8	21.0
7.1.1990	10.00 a.m.	Inlet	8.0	18.1
		Oxidation ditch	7.9	19.4
		Outlet	7.9	19.3
	3.00 p.m.	Inlet	7.7	18.8
		Oxidation ditch	7.9	20.8
		Outlet	8.1	21.4
8.1.1990	10.00 a.m.	Inlet	7.8	17.9
		Oxidation ditch	7.8	19.2
		Outlet	8.0	19.1
	3.00 p.m.	Inlet	7.8	18.9
		Oxidation ditch	7.8	20.8
		Outlet	7.9	20.6
9.1.1990	10.00 a.m.	Inlet	7.8	18.3
		Oxidation ditch	7.9	19.3
		Outlet	8.1	19.2
	3.00 p.m.	Inlet	*	*
		Oxidation ditch	*	*
		Outlet	*	*
10.1.1990	10.00 a.m.	Inlet	7.6	18.4
		Oxidation ditch	7.7	19.4
		Outlet	8.0	19.8
	3.00 p.m.	Inlet	7.8	19.3
		Oxidation ditch	8.0	20.9
		Outlet	8.0	21.1
11.1.1990	10.00 a.m.	Inlet	7.4	18.7
		Oxidation ditch	7.7	19.8
		Outlet	7.8	19.9
	3.00 p.m.	Inlet	7.8	19.5
		Oxidation ditch	7.7	21.8
		Outlet	7.8	22.2

APPENDIX 3 (1/2)

DO and temperature data.

Date	Time	Measuring points									
		1		2		3		4		5	
		DO mg/l	Temp. °C	DO	Temp.	DO	Temp.	DO	Temp.	DO	Temp.
5.1.1990	11.00 a.m	0.1	18.8	0	18.8	0	18.8	0	18.8	0	18.8
	1.00 p.m	0.1	19.4	0.1	19.4	0.1	19.4	0.1	19.4	0.1	19.4
	3.00 p.m	0.2	19.9	0.2	20.0	0.2	20.0	0.2	20.2	0.2	20.0
6.1.1990	11.00 a.m	0.1	19.3	0.1	19.3	0.1	19.3	0.1	19.3	0	19.2
	1.00 p.m	0.1	19.8	0.1	19.7	0.1	19.8	0.1	19.8	0.1	19.8
	3.00 p.m	0.2	20.3	0.2	20.3	0.2	20.3	0.2	20.3	0.2	20.3
7.1.1990	11.00 a.m	0.1	19.4	0.1	19.5	0.1	19.4	0.1	19.4	0.1	19.4
	1.00 p.m	0.2	20.4	0.2	20.1	0.2	20.1	0.1	20.0	0.1	19.8
	3.00 p.m	0.1	20.8	0.1	20.6	0.1	20.6	0.1	20.6	0.1	20.5
8.1.1990	11.00 a.m	0.3	19.6	0.2	19.3	0.3	19.3	0.3	19.3	0.2	19.3
	1.00 p.m	0.2	20.6	0.2	20.2	0.2	20.1	0.2	20.1	0.2	19.9
	3.00 p.m	0.3	20.8	0.1	20.5	0.2	20.6	0.2	20.5	0.2	20.5
9.1.1990	11.00 p.m	0.1	19.6	0.1	19.4	0.2	19.3	0.2	19.3	0.2	19.2
	1.00 p.m	0.3	21.1	0.3	20.2	0.3	20.2	0.2	20.3	0.2	20.1
	3.00 p.m	0.2	21.7	0.2	20.8	0.2	20.8	0.2	20.8	0.2	20.8
10.1.1990	11.00 a.m	0.2	19.7	0.2	19.7	0.2	19.6	0.2	19.6	0.2	19.6
	1.00 p.m	0.2	20.9	0.2	20.4	0.2	20.4	0.2	20.4	0.1	20.4
	3.00 p.m	0.2	20.7	0.2	20.8	0.2	20.8	0.1	20.8	0.2	20.7
11.1.1990	11.00 a.m	0.1	20.2	0.1	19.9	0.1	19.9	0.1	19.8	0.1	19.7
	1.00 p.m	0.2	20.8	0.2	20.6	0.2	20.6	0.2	20.6	0.1	20.5
	3.00 p.m	0.2	21.5	0.2	21.3	0.2	21.3	0.2	21.2	0.1	21.0

Date	Time	Measuring points									
		6		7		8		9		10	
		DO mg/l	Temp. °C	DO	Temp.	DO	Temp.	DO	Temp.	DO	Temp.
5.1.1990	11.00 a.m	0	18.8	0	18.8	0	18.8	0	18.8	0	18.8
	1.00 p.m	0.1	19.4	0.1	19.4	0.1	19.4	0.1	19.4	0.1	19.4
	3.00 p.m	0.2	20.0	0.2	20.0	0.2	20.0	0.2	20.0	0.2	20.0
6.1.1990	11.00 a.m	0.1	19.2	0	19.1	0	19.2	0	19.2	0	19.1
	1.00 p.m	0.1	19.8	0.1	19.8	0.1	19.8	0.1	19.8	0.1	19.8
	3.00 p.m	0.2	20.3	0.2	20.3	0.2	20.3	0.2	20.3	0.2	20.3
7.1.1990	11.00 a.m	0.1	19.3	0	19.4	0.1	19.4	0.1	19.4	0	19.3
	1.00 p.m	0.1	20.0	0.1	20.0	0.1	20.0	0.1	20.1	0.1	19.8
	3.00 p.m	0.1	20.5	0.1	20.5	0.1	20.6	0.1	20.6	0.1	20.2
8.1.1990	11.00 a.m	0.2	19.3	0.2	19.2	0.2	19.3	0.2	19.3	0.2	19.3
	1.00 p.m	0.2	19.9	0.2	19.9	0.2	19.9	0.2	19.9	0.2	19.9
	3.00 p.m	0.2	20.5	0.2	20.4	0.2	20.6	0.2	20.6	0.2	20.5
9.1.1990	11.00 a.m	0.2	19.2	0.2	19.3	0.2	19.3	0.2	19.3	0.1	19.2
	1.00 p.m	0.3	20.1	0.2	20.1	0.2	20.1	0.2	20.1	0.3	20.0
	3.00 p.m	0.2	20.7	0.2	20.8	0.2	20.8	0.2	20.7	0.2	20.5
10.1.1990	11.00 a.m	0.2	19.7	0.2	19.6	0.2	19.7	0.2	19.7	0.2	19.7
	1.00 p.m	0.1	20.5	0.1	20.4	0.1	20.5	0.1	20.5	0.1	20.5
	3.00 p.m	0.2	20.8	0.1	20.6	0.1	20.8	0.1	20.7	0.1	20.8
11.1.1990	11.00 a.m	0.1	19.9	0.1	19.8	0.1	19.9	0.1	19.9	0.1	19.8
	1.00 p.m	0.2	20.7	0.2	20.7	0.1	20.7	0.2	20.6	0.2	20.6
	3.00 p.m	0.1	21.3	0.1	21.3	0.2	21.3	0.1	21.3	0.1	21.3