WASTE STABILISATION POND SEWAGE TREATMENT SYSTEM: ITS APPROPRIATENESS TO A REGIONAL CITY OF THAILAND

BY

YANYONG INMUONG, B.Sc.

SUBMITTED IN PARTIAL FULFILMENT OF THE REQUIREMENT FOR THE DEGREE OF MASTER OF ENVIRONMENTAL STUDIES
(BY COURSEWORK)

CENTRE FOR ENVIRONMENTAL STUDIES
DEPARTMENT OF GEOGRAPHY AND ENVIRONMENTAL STUDIES
UNIVERSITY OF TASMANIA
NOVEMBER 1993

This thesis contains no material which has been accepted for the award of any other degree or diploma in any other University, and to the best of my knowledge contains no copy or paraphrase of materials previously published or written by another person, except when the reference is made in the text.

Y. Inmuong

Yanyong Inmuong
University of Tasmania
November 1993

ACKNOWLEDGEMENTS

This study is an original document attempting to find out the appropriateness of waste stabilisation ponds (WSPs) to Thailand. I would be very grateful if the results of this study could be utilised for the development of a biological sewage treatment system in any regional city.

Many people provided support in both Thailand and Australia. I would like to extend grateful thanks to my supervisor, Dr Peter McQuillan, who was very kind and provided regular encouragement during the study. He also devoted his time to assisting me in Thailand during the field work operation.

Dr Wanpen Virojanakoot from the Department of Environmental Engineering, Khon Kaen University, was very important to me. Valuable advice and supply of equipment were willingly offered by her, as a contribution towards the future development of this biological wastewater system in Thailand.

Documents and suggestions about tropical WSPs were given by Dr Boonsong Kaikate, Department of Environmental Health Science, Mahidol University for which I am indebted.

Associate Professor Srisumon Sitathanee of the Department of Biology, Khon Kaen University aided me in identifying the algal species. Her generous contribution is very much appreciated.

Staff of the Water Supply Regional Centre 6, who supplied the COD tests throughout the study, all the staff from my office, Environmental Health Centre Region 6 in Khon Kaen, who helped and inspired me, my director who provided laboratory equipment and a vehicle until the field project was finished should be acknowledged. Without these people this study would be incomplete, my sincere thanks to them.

My great editors, Jane Atkinson and Yolanda R. Robles who worked patiently with me are gratefully acknowledged in this piece of work. Due appreciation is also extended to the staff at the Centre for Environmental Studies and classmates who inspired and contributed valuable comments.

To my wife, my son and daughter, who are living a thousand miles away and who were patiently waiting for their husband and father, this thesis is dedicated. Finally, thanks to Mother Earth who cradled me in her arms and for which this humble work would be valuable.

ABSTRACT

An investigation of Khon Kaen WSP revealed that the WSP was more appropriate to the Thai regional cities' situation in both sewage treatment objective and socio-economic context. The WSP was capable of treating raw sewage meeting Thailand's effluent standard in BOD, pH, total coliform and faecal coliform counts, except TSS.

The effluent quality of Khon Kaen WSP had physical and biochemical average values of BOD as 14.8 mg/L, COD 83.0 mg/L, TS 452.2 mg/L, TDS 401.0 mg/L, TSS 51.5 mg/L, NH_3-N 0.5 mg/L, NO_3-N 0.2 mg/L, NO_2-N 0.3 mg/L, PO_4-P 0.0 mg/L, DO 6.5 mg/L and pH 7.5. Average bacterial counts in the final discharge were: total coliform of 120.7 MPN/100 ml and faecal coliform of 48.3 MPN/100 ml. The Khon Kaen WSP had high efficiency in reducing BOD (80.1%), NH_3-N (97.1%), PO_4-P (100%), total coliform (99.9%) and faecal coliform (99.9%). However, the plant was unable to reduce TS, TDS, TSS, NO_2-N and NO_3-N. Low PO_4-P and NO_3-N levels were discharged from the plant (even though the plant was ineffective in decreasing the NO_3-N level) and no eutrophication was induced in the receiving stream.

Sunlight, temperature, pH, phosphate, nitrate, retention time, pond geometry, pond location towards the wind direction and removal of algal mats appeared to be major factors influencing the biological pollutant degradation activities inside the ponds. Offensive odour, as a main public complaint about the plant, resulted from the abundance of floating algal mats.

Some conventional WSP design equations were found to be less effective in effluent quality prediction compared with the actual pond performance. The result of the analysis of some of the contemporarily designed BOD loading equations applied to this plant showed that such equations were less appropriate to the Khon Kaen city environment. The actual BOD loading-removal linear equation investigated from this existing plant could be the best method for future WSP development in this region.

The plant, with low costs of construction, operation and maintenance, was found to be cost-effective and suited to Khon Kaen city conditions. The plant required more operators and the present workers need additional training and experiences. Supplying the operators with equipment for removing algal mats appeared to reduce offensive odours and increased pond performance. Due to TSS level in the final discharge being over the country's standard, initiating any measure for decreasing TSS would be necessary for this plant.

(Note: for all abbreviations see pages x-xi)

TABLE OF CONTENTS

TABLE OF CONTENTS (cont.)

LIST OF TABLES

LIST OF TABLES (cont.)

LIST OF FIGURES

LIST OF FIGURES (cont.)

LIST OF FIGURES (cont.)

LIST OF PICTURES

ABBREVIATIONS

NAMES

AIDAB	Australian International Development Assistance Bureau
APHA	American Public Health Association
BMO	Bangkok Metropolitan Office
DITAC	Department of Industry, Technology and Commerce
DOH	Department of Health
DPC	Department of Pollution Control
KKCC	Khon Kaen City Council
KKM	Khon Kaen Municipality
KKPO	Khon Kaen Provincial Office
KKU	Khon Kaen University
MOI	Ministry of Interior
MPH	Ministry of Public Health
OKA	Office of the King's Affairs
ONEB	Office of National Environment Board
ONSEDB	Office of National Social Economic and Development Board
TDRI	Thailand Development and Research Institute
TLGA	Thailand Local Government Association
TISTR	Thailand Institute of Scientific and Technological Research
USA	The United States of America
UNDP	United Nations Development Programme
WHO	World Health Organisation

UNITS

^0C, ^0F	degree Celsius, degree Fahrenheit
cu.m	cubic metre
d, d^{-1}	day, per day
g/ca/d	gram per capita per day
ha	hectare
kg/d	kilogram per day
kg BOD_5 /ha d	kilogram of biochemical oxygen demand-5 days per hectare per day
lbs	pounds
m, m^2, m^3	metre, square metre, cubic metre
mg/L	milligram per litre
MPN/100 ml	most probable number per 100 millilitre
^0N	degree north
T	temperature

OTHERS

AL	Aerated lagoon
AN	Anaerobic pond
AS	Activated sludge
ASP	Activated sludge plant
BOD	Biological oxygen demand
CO_2	Carbon dioxide
COD	Chemical oxygen demand
DO	Dissolved oxygen
EA	Extended aeration
FC	Faecal coliform
FP	Facultative pond
HRT	Hydraulic retention time
H_2S	Hydrogen sulfide
ITP	Imhoff tank plant

ABBREVIATIONS (cont.)

OTHERS

MP	Maturation pond
N	Nitrogen
NH_3-N	Ammonia-nitrogen
NO_2-N	Nitrite-nitrogen
NO_3-N	Nitrate-nitrogen
OD	Oxidation ditch
P	Phosphorus
pH	Hydrogen ion concentration
PO_4-P	Phosphate (ortho)
PTP	Primary treatment plant with separate sludge digestion
RBC	Rotating biological contactor
SS	Suspended solid
TC	Total coliform
TDS	Total dissolved solid
TF	Trickling filter
TFP	Trickling filter plant with separate sludge digestion
TOC	Total organic carbon
TSS	Total suspended solid
WSP	Waste stabilisation pond

CHAPTER 1

INTRODUCTION

1.1 Background

Water pollution problems have become a major issue of public concern in Thailand. These troubles have two major sources: (1) sewage discharged into rivers due to an increasing urban population, with particular problems occurring in large cities, and (2) wastewater effluent from factories, which tend to locate on the river banks (ONSEDB 1992). The Thai population is still affected by water-borne diseases such as salmonellosis and amoebiasis which often result from drinking water polluted by discharged wastewater and transmitted into human gastro-intestinal tracts (DOH 1991). The 1992 data for the occurrence of diarrhoeal diseases in the country revealed that such diseases have a morbidity rate of 1,315 cases per 100,000 persons and a mortality rate of 1.05 cases per 100,000 persons (MPH 1993).

The wastewater problem in Thailand mainly results from the country's economic expansion, particularly in the large cities (Ludwig 1987). Water pollution crises occur mostly in the main rivers which are running through the big cities and have high levels of biochemical oxygen demand (BOD), and low levels of dissolved oxygen (DO) (Ludwig 1992). The Department of Pollution Control of Thailand (DPC 1992) noted that a city sewage treatment plant would play a vital role in alleviating water pollution problems in those rivers.

To help improve this situation, the Thai government has established a water quality standard which was legalised and imposed in 1986 (ONEB 1989). Major cities all over the country have been forced by the central government to establish sewage treatment plants (DPC 1992). Consequently, several technologies for treating municipal sewage have been recently introduced to various cities. These are mainly conventional treatment methods such as activated sludge, oxidation ditch, trickling filter, and rotating biological contactor (DPC 1992). Such sewage treatment systems require equipment and expertise from developed nations (TISTR 1989), but the high cost of these imported technologies is a serious problem (MOI 1989).

Appropriate low cost technologies for purifying the city sewage are, therefore, of major interest to some of the bigger city councils such as Khon Kaen, Nakorn Ratchasrima in the northeast and Supanburi in the central part (MOI 1990). One appropriate technology is the Waste Stabilisation Pond (WSP), which is claimed to be low cost in terms of construction, operation and maintenance (TISTR 1989).

The Khon Kaen City Council was the first council favouring this type of system and proposed a programme of WSP systems for central government financial support. The central government, whose budget is supported by the World Bank, agreed to the Khon Kaen WSP project and the first WSP sewage treatment plant in Thailand began construction in 1987. It was completed in 1989 and has been in service purifying Khon Kaen Municipality wastewater since then (KKM 1992).

Nevertheless, there are some potential problems arising from the conventional WSP system, including:

(1) the WSP system cannot always purify municipality sewage effectively to meet the effluent standard such as in USA (Hill and Shindala 1977, Mackay 1978, Weigand 1983, McKinney 1990) and in Nigeria (Okoronkwo and Odeyemi 1985),

(2) the WSP process is sensitive to local conditions and the internal processes of WSP are not yet fully understood (Mara 1976, Bredley and Silva 1977, Middlebrooks *et al.* 1979, Dinges 1982, Pescod and Mara 1988),

(3) there are various design models for WSP systems, and there are some points still in contention among them such as depths and shapes of WSPs which quite often lead to a question of which models would provide more effective results in biological sewage treatment mechanisms (Middlebrooks *et al.* 1979, Ellis 1983, Buchberger *et al.* 1989, Yhdego 1992),

(4) complaints are often made by the public due to inadequate performance of the WSP system (Ellis 1983 and Hess 1983).

Since there has been no study on functions, performances and the above mentioned problems of the municipal WSP sewage treatment system in Thailand, it is planned in this study to investigate the WSP sewage treatment system, which has just been constructed in one of the regional cities in Thailand, Khon Kaen City.

The water pollution control policy of the Thai government will be briefly mentioned. Wastewater conditions and a history of Khon Kaen WSP, the selected study site, also need to be clarified, since such factors are closely related to the WSPs' application. Reviews of the theory and practice of WSP systems from several regions in the world are essential as such data will provide a comparative background to this study.

1.2 Objectives of this study

It is my intention to test the Khon Kaen WSP, as a case study, to determine whether it is appropriate to the conditions of a typical regional Thai city or not. The objective of this study is to test the appropriateness of this WSP to the regional Thai city conditions. The following topics will be examined:

(1) the efficiency of the WSP in dealing with city sewage purification in compliance with the country's effluent standards,

(2) whether the function of the WSP is suitable to the local Thai environment,

(3) whether the design model used in this WSP (Khon Kaen WSP) can be put into practice,

(4) whether the WSP sewage plant performance has less emergent nuisances and,

(5) costs of construction, operation and maintenance and whether this type of treatment system is providing cost-effectiveness to the regional Thai city.

CHAPTER 2

THAILAND WASTEWATER MANAGEMENT SITUATION

2.1 The Thailand municipal sewage management scheme

The wastewater problems of Thailand have become increasingly critical following the rapid industrialised growth of the country in the past decade. The country's water pollution crisis has emerged as a result of urbanisation and enlargements of major cities, particularly Bangkok, the capital, and some large regional cities (TDRI 1987). One example is the Phong river which receives the municipal sewage discharged from Khon Kaen (one of the major business cities in Thailand). Prior to the construction of a WSP system, it was found that during the 1983 summer, average values of BOD and DO were 54 and 3 mg/L respectively, making the Phong river a risk to human and animal health (ONEB 1984). Significant amounts of PO_4 contributed by the Khon Kaen City sewage (averaging 4 mg/L) in this river also caused eutrophication (DOH 1985). Other rivers also encountered water quality problems. In 1984 the Chao Praya river had a BOD value of 61 mg/L and faecal coliform content greater than 4,000 MPN/100 ml.

However, the Thai government has addressed these water pollution problems since the introduction of the fifth national social economic plan (1982-1986). A major initiative in this plan involved the establishment of watercourse standards (ONSEDB 1982). As well, the central Thai government tried to encourage local governments to establish municipal wastewater treatment plants and provided budgets on the basis of 50 percent financial contribution (MOI 1984).

Most of the local governments complained about being unable to afford their share of the budget, since their funds were only sufficient for existing basic services, such as collecting rubbish and maintaining streets and they could not cope with a large project like a wastewater treatment plant (TLGA 1984).

Since 1984, with the serious budget constraint of local governments and an inability to construct sewage treatment plants, the central government has attempted to revise the national wastewater management programme by (MOI 1985a):

(1) lowering the amount of budget shared by the local governments from 50 to 30 percent,
(2) selecting some large cities facing urgent water pollution problems, to deal with as the first priority, and
(3) seeking alternative technology for municipal wastewater treatment.

The Khon Kaen Municipality was the first city to submit a sewage treatment plant proposal to the central government. It was subsequently selected to set up a municipal wastewater treatment plant. Of the various types of wastewater treatment technology available, the WSP was considered to be the system appropriate to Khon Kaen City (TISTR 1986a). However, the decision of the Khon Kaen City Council, which was recommended by TISTR, to select the WSP system, was primarily dependent upon its low cost of construction and ease of operation (KKM 1986). The efficiency of the treatment of municipal liquid wastes seemed to have minor priority. According to the Office of National Environment Board (ONEB) (1986), the WSP sewage treatment system required more systematic investigations before and after its operation, in terms of its efficiency, cost and potential problems. These examinations may later help the country, particularly other local governments, in selecting appropriate sewage treatment systems. Resulting from that recommendation, the concepts of ante and post evaluation are used when evaluating the WSP system in this study.

Initially, an attempt will be made to analyse systematically the possibility of using the WSP system for treating municipal sewage according to the Thai situation. The performance of WSP will be examined in the contexts of WSP energy, costs and treatment efficiency compared to other systems. Moreover, reviews of limiting factors and operation modes of WSP will also be presented in order to provide a complete overview of this system.

2.2 The WSP and other conventional sewage treatment systems

There are a number of sewage treatment system options available to city planners in Thailand, including activated sludge, trickling filter, oxidation ditch, and rotating biological contactor (ONEB 1986). These processes were developed by and for developed nations. Reid (1982) documented that in most developing countries that adopted conventional sewage treatment systems, the equipment and technology had to be imported from developed nations, which resulted in increased plant construction and operation costs.

This is true in the case of Thailand. In response to an examination of equipment and technology issues, the King of Thailand conducted research through a pilot study, installing a conventional sewage treatment system, the rotating biological contactor, at a temple in Bangkok aimed at domestic sewage purification. He found that 90 percent of equipment used had to be imported from Sweden. In addition, from plant installation to operation know-how, it was necessary to rely on expertise from overseas for the first six months (OKA 1984).

In terms of unit cost of treatment per cubic metre in that plant, he also found that the cost was very high, averaging $A 2.87 per cubic metre. Analysis of this cost showed it was mainly caused by energy demand, professional operator wages and equipment expenses.

High sewage plant construction costs are of major concern to the local Thai government. Such costs become a limiting factor for many municipalities when making decisions about the initiation of city wastewater treatment projects. The latest study conducted by the Department of Pollution Control, Thailand (1993) showed data from sewage plant feasibility studies in some cities of Thailand, in terms of plant construction and maintenance costs, as shown in Table 2.1.

Table 2.1 Comparative wastewater treatment plant costs in four cities in Thailand. (Source: Department of Pollution Control 1993)

Systems	Categories	Cost (1)	Cost (2)	Cost (3)	Cost (4)
Waste Stabilisation Pond (WSP)	Construction Cost	116,000	2,740,000	1,216,000	604,000
	Maintenance Cost	10,500	160,000	283,000	236,000
Oxidation Ditch (OD)	Construction Cost	198,000	3,990,000	2,389,000	-
	Maintenance Cost	34,000	475,000	703,000	-
Activated Sludge (AS)	Construction Cost	229,000	2,780,000	3,160,000	2,730,000
	Maintenance Cost	35,000	612,000	981,000	1,160,000
Rotating Biological Contactor (RBC)	Construction Cost	374,000	6,700,000	4,360,000	-
	Maintenance Cost	20,000	579,000	969,000	-
Aerated Lagoon (AL)	Construction Cost	-	-	-	612,000
	Maintenance Cost	-	-	-	665,000

Estimated cost ($ Aus) for wastewater treatment plants;

(1) Nonthaburi Municipality Zone 01 with sewage flow 1,000 m^3 per day, 1989;

(2) Pathumthani Municipality for discharged sewage of 10,000 m^3 per day, 1992;

(3) Ayutthaya Municipality with average discharge of 15,000 m^3 per day, 1992;

(4) Chiang Mai City with average sewage flow 87,000 m^3 per day, 1991.

From the above table, it is found that WSPs have a distinctly lower cost in both construction and operation modes than the other systems. With the capacity of BOD reduction at 85-95 %, low maintenance cost, no investment in sludge disposal required and no highly skilled labour required, city executives from all over Thailand are now interested in this system. Currently, some cities in the Northeast, Nakornrachasrima, Phrayao in the North and Supanburi to the West of the country are implementing a WSP programme to treat their municipal wastewater. Since there is no research into the performance of the Khon Kaen WSP plant, the first WSP in the country, this study is viewed as essential for contributing more information about WSPs to other local governments. This information includes the effectiveness of treating urban liquid wastes using a WSP in compliance with the National Effluent Standard as well as the cost effectiveness of this type of sewage treatment system.

The Department of Pollution Control (1993) viewed that WSPs will play a significant role in treating domestic sewage in the near future, especially in those cities which have limited alternatives due to the high cost of conventional sewage systems. This likelihood has been strengthened by The National Pollution Control Act 1992, which requires that all wastewater discharged into a natural waterway by industries and communities must be purified by a wastewater system. It is estimated that there will be nearly 400 cities (districts) suitable for WSPs particularly those small cities with low income. Those located in rural districts tend to have more land available for the WSP system, which also suits them in terms of its low cost and limited manpower needs (DPC 1993).

One such district, Kao Phra, is located in the remote west with a population of 9,147. A feasibility study into alternative sewage treatment plants found that this city was suited to WSP especially in terms of cost. The projected WSP cost is estimated at $A0.72 million for plant construction with operating and maintenance costs at $A 0.07 per cubic metre. To most city councils concerned, the plant operating cost of WSP represented the lowest expense compared to aerated lagoon and oxidation ditch systems, since such systems offered $A0.09 and $A0.13 per cubic metre respectively (TISTR 1993).

The high operation cost of more conventional sewage treatment plants, was reported by Piampongsan (1989) who investigated two existing small-scale plants in two suburbs of Bangkok. The suburb Bangkok Zone 05, uses the activated sludge process, supplying sewage treatment for 77,000 people, at an operating cost of $A5.76 per cubic metre. The suburb Bangkok Zone 17, serving about the same population size, uses an aerated lagoon system, with an operating cost of $A2.83 per cubic metre. Piampongsan (1989) also found that these two communities were encountering a problem meeting the high cost of plant operation and that they were seeking subsidies from the central government.

However, these two conventional systems were considered suitable for these communities, since they have high land costs.

The main difference of operation costs between WSP, activated sludge and aerated lagoon as TISTR (1993) and Piampongsan (1989) mentioned is mainly due to electricity costs. When the same system is used, for example aerated lagoon, but in different locations such as in Kho Pra district and suburb Bangkok Zone 17, the difference in operation costs per cubic metre comparing values from TISTR (1993) and Piampongsan (1989) is mainly dependent on the differences in each area's sewage. The Bangkok Zone 17 has a high economic profile and thus it produces higher BOD and lower amounts of oxygen in its sewage whilst Kho Pra is only a small rural town and produces less BOD content and higher levels of oxygen. For this reason, the time needed to run aerators in Bangkok Zone 17 is much greater than in Kho Pra and thus makes it more costly.

A review of the relative plant costs of WSP and other conventional sewage plants in Asia can be found in Reid (1982). From his document it is clear that WSPs still represent the lowest cost in both construction and operation (Table 2.2).

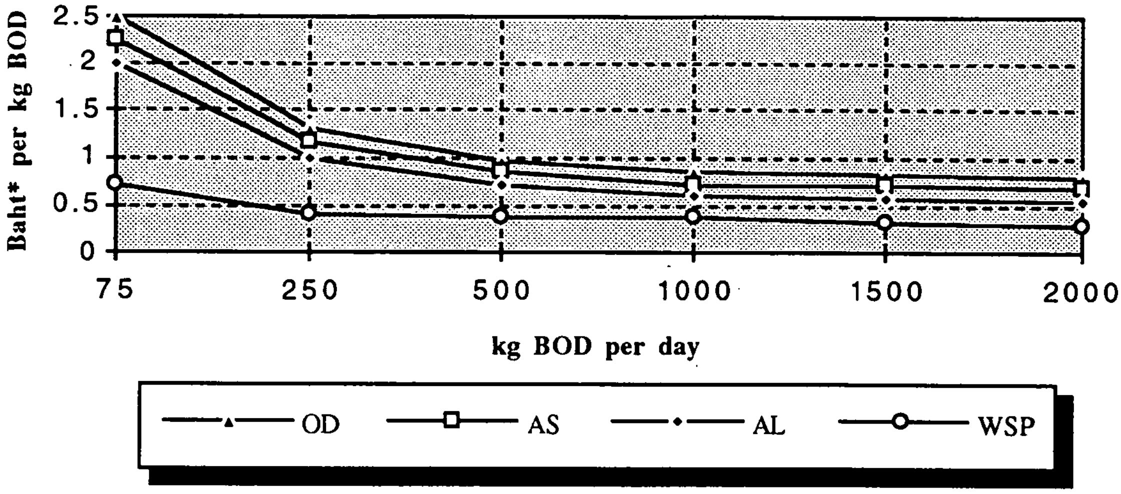
However, the estimated construction cost per capita of WSP in Table 2.2 (Reid 1982) is low ($A2.06-cost in 1989) compared to the Khon Kaen WSP system's cost ($A7.73-cost in 1989). The operation and maintenance cost per capita of this plant in 1992, was also considerably low at $A0.25, while Reid's cost was $A0.66. Reid's latter costing applied to a population of 100,000, approximately similar to that served by the Khon Kaen WSP (109,241) and that cost is adjusted to 1992 costs by using price indices. Table 2.2 also showed other conventional systems having a higher cost of plant operation compared to WSP, due to electricity costs.

Table 2.2. Estimated cost of wastewater treatment plants in Asia. (Source: Reid 1982)

Type of treatment process	Design for population size	Design flow in (million gallons per day)	% Cost of imported wastewater equipment	Estimated construction cost per capita ($)	Estimated operation and maintenance cost/capita/yea-r ($)
Stabilisation Pond	5,000	0.15	-	3.27	0.54
	10,000	0.30	-	2.81	0.47
	50,000	1.50	-	1.99	0.35
	100,000	3.00	-	1.71	0.31
	200,000	6.00	-	1.48	0.27
Aerated Lagoon	5,000	0.15	-	4.00	1.22
	10,000	0.30	-	3.59	1.15
	50,000	1.50	-	2.79	0.98
	100,000	3.00	-	2.50	0.92
	200,000	6.00	-	2.25	0.86
Activated Sludge	5,000	0.15	25	10.99	2.92
	10,000	0.30	25	9.00	2.17
	50,000	1.50	25	5.65	1.09
	100,000	3.00	25	4.62	0.81
	200,000	6.00	25	3.79	0.61
Trickling Filter	5,000	0.15	25	12.09	3.66
	10,000	0.30	25	9.55	2.88
	50,000	1.50	25	5.51	2.58
	100,000	3.00	25	4.33	2.29
	200,000	6.00	25	3.43	1.00

Rattanasuk and Grinsukon (1981) compared the purification cost per Kilogram BOD of four sewage treatment systems located in large factories in Thailand. They found that the WSP was able to provide the least cost of treatment per Kilogram BOD compared to the other processes (Figure 2.1). The high expense of the oxidation ditch, activated sludge and aerated lagoon is mainly due to energy costs and professional operators' wages. This is a similar result to Canter *et al.* (1982) who investigated conventional sewage treatment plants and concluded that conventional plants are not only energy-intensive, but also needed skilled operators.

Figure 2.1 Comparison of budget requirements between four sewage treatment systems in Thailand.
(Source: Rattanasuk and Grinsukon 1981).



1 A$ = 17 Baht*

Arthur (1983), a sewage treatment expert of the World Bank, evaluated 11 plants of WSPs and ALs operating in developing countries and found that WSPs had a lower operating cost compared to aerated lagoons (AL) (Table 2.3). The WSP processes employed natural oxygenation reaeration whereas ALs used mechanical aerators for supplying oxygen to the pond. The ALs, therefore, led to a high operation cost due to power consumption.

Table 2.3. Comparative operation costs of WSP and AL systems in five developing countries.
(Source: Arthur 1983)

Location	Total operation cost ($)	Operation cost per capita ($)	Type of system
Dagat Dagatan (Philippines)	92,300	2.05	AL
Netanya (Israel)	260,000	2.60	AL
Yavneh (Israel)	100,000	6.30	AL
Eilat (Israel)	40,000	1.33	AL

Dadora (Kenya)	5,200	1.33	AL
Ngwerere (Zambia)	9,000	0.25	WSP
Manchichi (Zambia)	11,000	0.11	WSP
Matero (Zambia)	9,000	0.30	WSP
Chelston (Zambia)	3,500	0.18	WSP
Munali (Zambia)	2,300	0.12	WSP
Spanish Town (Jamaica)	5,500	0.61	WSP

The cost of construction and maintenance of WSPs is found to be dependent upon the country's social and economic situation. Reid and Muiga (1982) analysed construction and maintenance costs of WSPs in the USA and India. They found that the construction cost per capita of a WSP in the USA was higher than that of India, but the maintenance cost per capita was similar in these two countries. This can be seen in Table 2.4 and would suggest that WSPs are cheaper to construct in developing countries than in richer nations.

Table 2.4 Cost comparison of WSPs between the United States of America and India.
(Source: Reid and Muiga 1982)

Population	United States	of America	India	
	Construction ($/Capita)	Operation and Maintenance ($/Year/Capita)	Construction ($/Capita)	Operation and Maintenance ($/Year/Capita)
5,000	16.56	0.50	2.09	0.32
10,000	10.89	0.39	1.84	0.25
25,000	-	-	-	-
50,000	4.11	0.20	1.29	0.17
100,000	2.70	0.14	1.25	0.14
200,000	1.78	0.11	1.17	0.12

Note: land costs are excluded.

Another problem associated with conventional sewage treatment plants is the professional manpower requirement. Looking at skilled sewage treatment manpower availability in Thailand, it is found that there are only three schools of environmental engineering in the country, at Chulalongkorn, Chiang Mai and Khon Kaen Universities. The number of students graduating each year is very low compared to the country's requirement. In 1992 only 17 graduated (ONEB 1993) and these graduates prefer private sector jobs as plant operation engineers, controlling wastewater treatment plants in large factories. At present none are working in the public sector. Consequently, the lack of skilled persons who are capable of operating the conventional sewage systems, poses problems for any municipality attempting to develop such systems in its city.

Reid (1982) documented that WSP is the only sewage treatment system which needed less skilled manpower and resources, while the conventional plants, for instance activated sludge and rotating biological contactor, require highly skilled operators, imported equipment as well as chemical supplies.

Sludge disposal from the conventional sewage treatment plant is another problem with the most common system e.g. the activated sludge process. Canter *et al.* (1982) noted that most of the plants in operation, normally encountered a sludge disposal problem. When considering sludge treatment, Bridle (1991) stated that the disposal of sludge from such plants accounted for up to 50 percent of the total plant operating costs and that this process needed professional manpower. Sludge treatment processes in general involve a process of thickening, dewatering and drying. This process presents problems in the cities where wastewater treatment schemes have been recently developed. The cost of sludge disposal equipment, like a dissolved air flotator, centrifuge or rotary screen thickeners, are currently very high and they are only globally supplied by multinational companies (Gobbie 1990).

Therefore small scale cities in rural districts of Thailand and other developing countries find it virtually impossible to apply this kind of conventional system. Metcalf & Eddy Inc (1979) noted that one of the advantages of the WSP system is the sludge disposal operation, since this system needs no routine sludge disposal works, and only requires sludge removal from the pond bottom every 5-10 years depending on the type of pond used.

Another concern affecting the various conventional wastewater treatment plants is what constitutes domestic liquid waste. It has a high content of human waste and consequently pathogenic bacteria. It has been shown that conventional systems cannot purify domestic liquid wastes, rendering them micro-pathogen safe unless the final treated effluent is chlorinated (Dinges 1982).

Liquid effluent disinfection with chlorine is expensive and the Department of Pollution Control documented that in 1990 almost 50 percent of Thai districts could not cope with the expense. Further there were some concerns that carcinogenic compounds might be generated by liquid disinfected with chlorine, presenting a health hazard. Therefore, any sewage treatment system which does not require chlorine disinfection would be viewed as beneficial to Thai cities (DPC 1993).

The problem of nutrient reduction in domestic sewage is another issue considered in relation to conventional wastewater treatment plants. The activated sludge process, for example, is designed to reduce Biological Oxygen Demand (BOD). In other words, it aims to reduce the organic strength of sewage effluent to a level where it should not pollute the water course (i.e., causing oxygen depletion which would affect the stability of the aquatic ecosystem). The other purpose is to reduce Suspended Solids (SS) from the liquid discharged. This latter function is still regarded as unsatisfactory as it has often been found that nutrients such as phosphorus and nitrogen are at high levels in the effluent from conventional sewage treatment plants (Mara 1988). Methods of lowering these pollutant levels in conventional plant effluent are currently being researched. With the growing acknowledgement of the need for improved environmental quality, nutrient removal technology is becoming a popular area of research. This nutrient reduction know-how is considered to be the most significant research topic in the 1990s (DITAC 1990). Some findings from completed studies have already been converted into sewage treatment commercial operations.

One recent example of technology for improving effluent quality from conventional plants, especially *E.coli* and nutrients, is that of membrane technology using micro and nano filtration processes. However, the cost of filter cartridges used in the membrane system is high.

Another example is the patented process known as the Alternating Aerobic/Anaerobic Activated Sludge (AAA) process, developed by CSIRO's Division of Chemicals and Polymers, Australia. This can reduce nitrogen and phosphorus content in treated effluent to meet the required effluent standard (O' Gallagher 1990). Another alternative system for reducing nutrients is using WSPs as polishing ponds in combination with conventional units. It is considered that this pond system consumes less energy and costs less. This combined system has been used to improve nutrient levels meeting the effluent standard in, for example, Gumeracha City, Adelaide (Mitchell and Williams 1982) and the Union Carbide Company, Texas, USA (Martin *et al.* 1976). The ability of phytoplankton to take up nutrients, which are the product of microorganisms breaking down organic matter in a WSP, shows that WSP is another optional system for improving nutrient levels in the effluent up to the standard.

These technological innovations have mainly resulted from the introduction of more stringent environmental standards in many countries requiring higher quality effluent.

An example of the detrimental impact caused by inefficient nutrient reduction in the conventional sewage treatment plant, and the high cost required for its improvement can be seen from Tasmania. Recently residents of the cities of Clarence and Sorell, Tasmania, Australia, have been facing the problem of algal blooms and anoxic conditions in natural waterways. This has been caused by the discharge of inadequately treated effluent from conventional sewage plants. The local and central governments now have to spend a large amount of the cities' budget upgrading two activated sludge treatment plants. To improve the efficiency of Rosebay sewage treatment plant Clarence City requires $A1.9 million, and Sorell city plans to spend $A1.5 million for the same reason (The Mercury, 13 July 1993).

Thus, the conventional sewage treatment plant, such as the activated sludge type, is not proving to be the most effective device due to the high content of nutrients still present in treated effluent. This conventional plant still requires additional processes to remove nutrients, especially nitrate and phosphorus. These additional processes tend to be costly and involve high technology. So, it seems that as well as problems of high cost involved with conventional plants there is a problem of inefficient nutrient reduction. These problems should be considered in the context of Thai regional cities when determining what kind of sewage plant they should adopt.

When selecting the appropriate sewage treatment technology, Thai city authorities need to consider the country's wastewater treatment policy. The Thai Environment Act 1985 has forced city councils to introduce systems which will enhance the national wastewater treatment goal. It would also be more beneficial to individual cities if the chosen system is appropriate to the local socio-economic context. The local authorities in cooperation with central government now see the value of undertaking a feasibility study of possible sewage plants for each city. In this way they can determine their requirements in terms of national policy and local conditions (DPC 1993). This would include land use planning for sewage treatment purposes if the WSP has been proved to be an effective means of waste treatment in that city.

To date the country's wastewater treatment development goal, which was set up in 1989, is primarily directed at protecting community health (ONEB 1989). Secondly its aim is to cause the purification of contaminated sewage so as to minimise the damage to waterways. The first strategy is to control pathogens, and the second approach is to remove oxygen-demanding materials and suspended solids (Suphapodhog 1991).

This national goal is similar to that of developed countries during 1964-1972, while the current trend in these nations aims towards uncontaminated effluent from all discharges (Canter *et al.* 1982). The Thai ultimate goal, to reduce pathogenic bacteria, BOD and nutrients, is not expected to be attained for at least 20 years and is conditioned by the kind of sewage treatment system applied. This time scale could be shortened if the cities can soon find the most appropriate method for treating their wastewater (DPC 1993). For some cities the WSP system may be favourable and its application is being investigated in this study.

To determine the feasibility of systems to treat regional municipality wastes, it is found that international agencies have a prime role to play. For example, the Australian International Development Assistance Bureau (AIDAB), Bangkok office, has made contributions to studies in Thai regional cities (MOI 1989). However, this organisation still has limited capacity since so many cities are requesting assistance, the present number being 112 cities, while AIDAB can provide assistance to only 2-3 municipalities per year (AIDAB-BANGKOK 1989).

Due to the above constraint, the Thai government is encouraging overseas consultant companies to contribute also. But, due to the high cost of these international firms there are only 15 cities which have conducted feasibility studies, and still the majority cannot afford to do so. From the completed feasibility studies, the results reveal that 12 of the cities would be suited to the WSP system (DPC 1992a). Nevertheless, such studies only provide a general picture of a given city's wastewater master plan that should be adopted, and they do not include any details of the WSP system to be constructed. This further exercise requires more expense. At present, there are only 2 cities that can afford the expense and have completed the design stage of a WSP: Prayoa (61,000 dwellers) and Skornakorn (53,000 population) (DPC 1993). It is interesting to note that among these studies, no Thai city has studied the biochemical mechanisms of the WSP.

The National Effluent Standard, established in 1989, is the first attempt by the Thai government to deal with municipal wastewater discharge. The standard itself, as shown in Table 2.5, still needs further input regarding certain values of nitrogen and phosphate content. The initial objective of the Standard is to prioritise BOD_5, solids, pH, total coliform and faecal coliform bacteria, and this standard is seen as the primary stage to be imposed. In addition, the central government viewed that this standard would alleviate the country's water pollution problems as a whole and that communities in general should apply this standard (DPC 1992b). In response to this required standard, all municipalities are currently trying to develop their sewage plant schemes, as mentioned earlier. The values of BOD_5, solids, pH, total coliform and faecal coliform bacteria are taken as the primary consideration among the municipalities.

However, since the future trend of this standard will be to include nutrient (nitrogen and phosphorus) discharge values, then most of the city managers are including those criteria when judging sewage treatment alternatives (TLGA 1991).

Table 2.5 The Thai community sewage effluent standard.
(Source: Department of Pollution Control 1992b).

Qualified Items	Units	Values	Remarks
1.Biochemical Oxygen Demand	mg/l	20	
(BOD)			
2.Solids			
2.1 Total Suspended Solid	mg/l	30	
2.2 Total Settleable Solid	mg/l	0.5	
2.3 Total Dissolved Solid	mg/l	500	
3.Nitrogen			* will be determined whenever
3.1 Ammonia Nitrogen	mg/l	*	public reservoirs polluted
3.2 Nitrate Nitrogen	mg/l	*	
4.pH	pH-value	5 - 9	
5.Total Coliform	MPN/100 ml	5000	
6.Faecal Coliform	MPN/100 ml	1000	
7.Phosphate	mg/l	*	

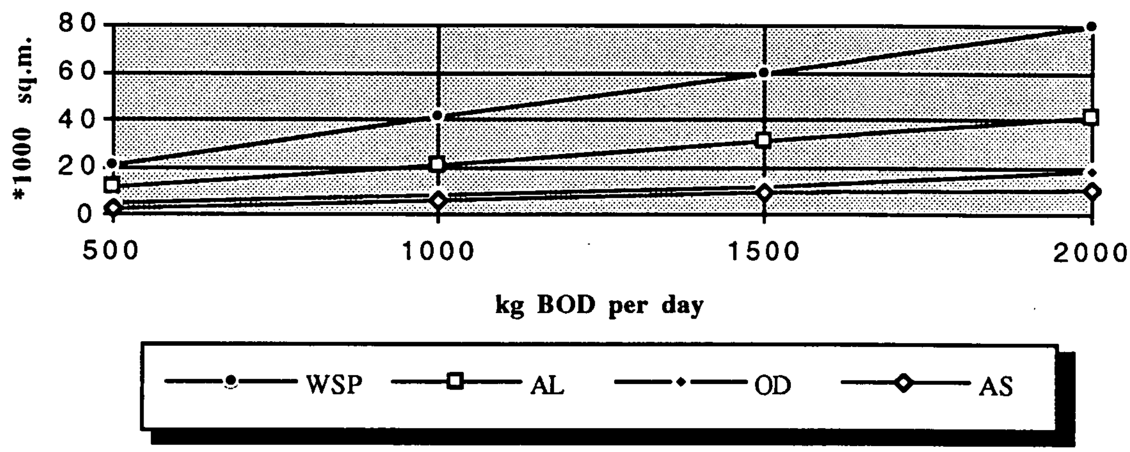
2.2.1 The question of land use

The WSP may often require greater areas of land than the other processes. A comparison of land requirements for four sewage treatment systems was made in Thailand by Rattasuk and Grinsukon (1981) who showed that WSPs required more land than activated sludge, oxidation ditch and aerated lagoon (Figure 2.2).

The Bangkok Municipality noted that due to the land requirement of WSP, the capital city would not be suited to this system, since land prices are very high (BMO 1987).

This situation is similar in big cities, including tourist towns such as Pattaya and Phuket. Thus, land availability is a potential limiting factor in developing WSP in certain Thai cities. This is a serious consideration which needs to be taken into account by Thai authorities when considering the feasibility of WSP wastewater schemes.

Figure 2.2 The relation between land requirement and BOD loading in some sewage treatment systems in Thailand.
(Source: Rattasuk and Grinsukon 1981)



Nevertheless, to a large extent, most Thai regional cities have some land available for public use. This is because, traditionally, most of the city councils reserve some land around their city for future development. These public lands are still set aside and mostly appear to be swamp areas in which substandard housing has been developed. Also, most of these lands are found to be breeding places for vectors. According to MOI (1989) such land is often suitable for a city sewage treatment programme and further it is estimated to be of sufficient area for any sewage treatment plant system if that city chooses to establish it.

The crucial land factor affecting WSP sewage treatment projects in some cities can therefore be minimised, although in some cities these reserves have been hired to commercial interests. The amount of land required for sewage treatment plant in each city still requires further technical planning research (DPC 1993).

2.2.2 Finances

Another crucial factor affecting the development of wastewater treatment plants is the municipalities' revenue. For instance, the annual budget of Khon Kaen Municipality is about 150,281,611 bahts (A$88 million), but only 10 % of this is spent on infrastructure development (KKM 1992b).

This situation is similar to other large cities and results in insufficient finance for necessary infrastructure projects including sewage treatment plants (MOI 1989).

With increasing environmental problems and with cities unable to afford sewage treatment plants, the central government aimed to support local government finances. However, the central government is also experiencing a financial crisis. Therefore the central government proposed a national wastewater treatment master plan to the World Bank. This programme was agreed to and supported by the World Bank with a soft loan and this money has contributed to the development of sewage treatment projects in some regional cities (MOI 1986).

The regional cities had to endure a further problem, that is the requirement to share the cost of initiating city sewage treatment projects. The central government will support local governments financially but the local governments have to meet at least 30 percent of the total sewage project expenses (MOI 1986). The other 70 percent is contributed by the World Bank through the central government. However, the majority of cities cannot afford their share of the money. There are two main reasons (1) the cities have less funds available than is required, and (2) even with the low interest rate of a soft loan, the city councils often consider the long term status of the city, particularly in terms of the potential financial burden to the next elected local government body (TLGA 1986).

However, there are some cities, like Khon Kaen in this study, Nakorn Ratchasrima and Supanburi which have agreed to shared budget condition and have signed the sewage plant development plan with the central government (MOI 1987). These cities have all selected the WSP system because of the system's low cost. The first large scale municipal WSP sewage treatment system in Thailand was established in Khon Kaen City, and later the same system was set up in Nakorn Ratchasrima and Supanburi Cities. Other regional cities are still waiting for a grant from the central government, because they are unable to afford the 30% cost share.

Recently, there has been a serious decline in natural environmental conditions in the southern and northeastern parts of the Kingdom. In 1989, the Thai government decided to revise the former Environmental Pollution Control Act 1986 and recently adopted a new law entitled The National Pollution Control Act 1992 (DPC 1992b). The National Environmental Fund was set up under that law, to support both government and non-government sectors deal with environmental problems. This is a source of finance available to local governments for the development of their sewage treatment system programmes. It is considered that such a fund is more appropriate to local government needs since it is more flexible in terms of low interest rates and extended duration of repayment than that of the World Bank source (TLGA 1993).

Nevertheless, the fund is insufficient to meet the needs of the various organisations requesting help. The central government therefore gives priority to the local governments of regional cities (Ministry of Finance, Thailand 1993). Still, since it is a loan and not a grant from the central government, 30% of local governments, as documented by DPC (1993) cannot repay the money to the central government according to the new fund conditions. This is happening mostly in small regional towns which have low incomes from local tax. However, at the moment, the central government has no measures to enhance the limited budgets of these cities. Therefore, it is more of a struggle for remote cities to develop sewage treatment schemes.

For the large regional cities, there are still some problems encountered even when their proposals for sewage treatment plants were accepted by the central government.

Firstly, if any city accepts the sewage treatment programme loan funding from the National Environmental Fund, municipal sewage treatment facility rates need to be established in order to repay the fund. This is proving difficult for city councils since there is no tax revenue available to spend on sewage treatment in any city of Thailand. Also referendum results have shown that each city household would not want to pay more than $A0.50 per month on such a tax, despite dwellers in the regional cities being in favour of the polluter pays principle (MOI 1991).

Secondly, because of city residents' aversion to a sewage tax, city councils are having less choice in selecting wastewater treatment options. Thus a city council is driven to choose any sewage treatment system that has a low cost of construction and maintenance. This is what determined Khon Kaen, Nakorn Ratchasrima, and Supanburi City Councils selecting the WSP.

2.3 Advantages of the WSP to the country's environment

There are some exclusive advantages of WSP, especially in the tropical and subtropical areas rather than the temperate zone. Benefits of the WSP system in tropical areas can be categorised into three main points; (1) the suitable climate, (2) benefits of effluent reuse and (3) high efficiency in reducing nutrients and others.

It has long been claimed that tropical and subtropical climates provide an ideal environment for natural sewage treatment systems, particularly the WSP process (e.g. in Dinges 1982, Arthur 1983, Pescod and Mara 1988, Tchobanoglous 1991). Raw sewage is detained in a series of shallow ponds, for say two to three weeks, and a significant level of both BOD and pathogen removal can be achieved.

The natural action of warmth and sunlight in tropical countries can promote the rapid growth of microorganisms which will efficiently react both aerobically and anaerobically to remove BOD and nutrients from raw sewage stored in the ponds (Tchobanoglous 1991).

During the 1980s the World Health Organisation (WHO) initiated a global programme titled "Water Supply and Sanitation Decade 1981-1990". This aimed to promote health for the world's population. Applying the appropriate technology for wastewater treatment was one of the proposed measures advocated for developing countries. Its member countries, particularly in Asia and the Mediterranean regions, were encouraged to try a low cost wastewater treatment WSP either by inventing a pilot scheme or developing a full scale WSP system for domestic wastewater treatment in their countries. WHO considered WSPs to be appropriate to these regions because (1) the climate of those countries is suitable and (2) WSP has a lower cost of construction and maintenance than the other systems (WHO 1981).

Bachmann (1980) concluded the benefits of applying WSP methods in developing countries can be summarised as follows;

- (i) ponds offer economic advantages for small and medium size communities,
- (ii) they require less energy,
- (iii) highly skilled operators are not necessary,
- (iv) effluents can meet the required standard, and
- (v) ponds are useful where no other receiving waters are available for the effluent and its re-use for land irrigation is possible.

Resulting from the WHO exercise, Pakistan, India, Jordan, Syria and Malaysia have initiated WSP projects in their countries (WHO 1981). However, there is limited documentation about WSP performance available from those countries. Even at the latest international conference on WSPs in Portugal 1989, when the application of WSP in tropical countries was considered, the only conclusion was that a WSP system was the most appropriate way of treating sewage in tropical countries.

Further, resulting from this seminar it was advised that tropical countries should study WSPs. Particular attention should be given to the issue of appropriate WSP models in terms of design, operation and performance suitable to the local environment. However, in Thailand, WSPs are still not the main interest among investigators. Most research is still in areas of industrial waste treatments rather than WSP.

Nevertheless, according to government policy mentioned earlier, the aim is to improve municipal wastewater treatments, so there are some investigations in this area. Due to a lack of sewage professionals, the creation of a project on WSPs and other municipal sewage treatment systems will need to be largely carried out by overseas firms (DPC 1993).

Table 2.6 shows a general summary of the advantages of WSP over other sewage treatment processes. For instance, when considering the water conservation issue, a WSP can perform a distinct benefit to regions in arid or hot zones. This was clearly evident from Israel (Arthur 1983). Irrigation and aquaculture using WSP effluent discharge has been demonstrated in some countries (Pescod and Mara 1988). Ahmed (1981) compared systems for BOD reductions. WSPs could perform 90% BOD removal, whereas trickling filter achieved 70-75%, with 85-90% for activated sludge. Disadvantages might emerge from WSPs in the case of (1) more land requirement and (2) high content of SS in treated effluent.

Table 2.6 Advantages and disadvantages of WSP compared to other sewage treatment systems.
(Source: Arthur 1983)

System	AS	TF	EA	OD	AL	WSP
Criteria						
BOD removal	**	**	**	***	***	***
FC removal	*	*	**	**	***	***
SS removal	***	***	***	***	**	**
Helminth removal	**	*	*	**	**	***
Virus removal	**	*	**	**	***	***
Ancillary use possibilities	*	*	*	*	***	***
Simple and cheap construction	*	*	*	**	**	***
Simple operation	*	**	*	**	*	***
Land requirement	***	***	***	***	**	*
Maintenance costs	*	**	*	*	*	***
Energy demand	*	**	*	*	*	***
Minimisation of sludge removal	**(b)	**(b)	**(b)	*	**	***
Effluent reuse possibilities	*(a)	*(a)	**	**	***	***

(a) Effluent from activated sludge and trickling filter frequently has high ammonia levels (>5 mg/l) and faecal bacterial concentrations. This is usually unsuitable for irrigation or fish farming without tertiary treatment.

(b) Assumes provision of sludge digesters.

Key: *** good; ** fair; * poor

WSPs are normally designed for BOD, SS and coliform bacteria reduction. The conventional effluent standard for WSP designs was aimed to diminish BOD, SS and coliform bacteria. The expected effluent levels of those three parameters are 20 mg/l, 30 mg/l and 5000 MPN/100 ml, respectively. Several documents ascertain that WSP has a good performance in meeting these standards, especially BOD and coliform reduction (Bachmann 1980, Hess 1983, Arthur 1983, Bucksteeg 1987, Pescod and Mara 1988). There are many conventional design models of WSP available, for example McGarry and Pescod (1970), Gloyna (1971) and Mara (1976). These conventional design formulae are still in use in several parts of the world. However, these WSP models require further investigation since these models do not work in some cases (Middlebrooks *et al.* 1979, Ellis 1983).

For World Bank supported programmes, as in Thailand, Arthur (1983) documented that WSP installed in the Philippines, India, Israel, Kenya, Zambia and Jamaica, displayed a wide diversity of designs used. But, these designs are applied from the western world. Further, Malone and Swann (1978) and Arthur (1983) found that WSP operators still needed more practical experience and information exchange on WSP performance.

In fact, the design of WSP has recently been developed. Many publications, seminars and workshops are still looking at new developments especially since the 1980s. These contributions from WSP research are mainly from the western world, only a few are from developing countries (Arthur 1983). Thus, much of the WSP design criteria applied in the developing world are still employing the western models.

Biological wastewater treatment experts agree that design models of WSP should include local environmental factors (McGarry 1982, Mara 1988). Moreover an assessment of existing WSPs within a country is viewed as essential for adjusting WSP design to local conditions. However, in Thailand, there is no such study yet available. The first WSP system at Khon Kaen, Thailand, simply employed western empirical formulae created by McGarry and Pescod (1970) and Marais (1974).

In conclusion, for the Thai wastewater management situation, the WSP system both generally and specifically might be suited primarily to wastewater treatment programmes of the regional cities. The WSP might be the most appropriate method when land cost is cheap. With problems of low incomes and scarce resources, the majority of small towns in the country might favour WSP with its advantages of low cost, ease of operation, no skilled operators needed, and high efficiency in domestic sewage purification. These would make WSP attractive to many regional city councils as a wastewater treatment device. WSP itself, on the other hand, requires more investigations in terms of its performance evaluation, factors influencing its effectiveness and the type of design model suitable to the local environment.

CHAPTER 3

THEORETICAL BACKGROUND AND LITERATURE REVIEW

3.1 History of waste stabilisation ponds (WSPs)

Waste Stabilisation Ponds (WSPs) represent one of the simplest forms of wastewater treatment processes and have been employed as community sewage purification systems for many decades. It was first documented as a municipal wastewater treatment system in the 1920s in California, North Dakota, and Texas in the USA (Middlebrooks *et al.* 1979, Dinges 1982). This type of man-made pond has also been used for community sewage treatment for centuries in Asia as Reid (1982) noted. Gloyna (1971) also reported that the Greeks adopted stabilisation ponds as fish ponds for hundreds of years.

Australia first started the WSP system in 1928 at Melbourne city (Parker, Jones and Greene 1959, Parker 1970, Seabrook 1975). This WSP has been used in treating sewage in conjunction with land filtration and grass filtration. The whole system is called the Werribee Treatment Complex. This treatment complex treats 55% of Melbourne's sewage (455,000 cu.m per day).

Conventional research on WSPs in the USA was first established in 1931 at Collidge Station in Texas. It was the study of the capability of an artificial lake in treating domestic sewage. This study was started after Giesecke visited Germany where he observed low dams built in the Ruhr River Valley receiving wastewater from the imhoff tank of a factory. Later, the innovation of WSP in Texas operated successfully in treating community sewage. Eventually, this plant was developed at a full-scale in 1933 (Dinges 1982).

As well as the successful application of WSP systems to treat sewage in some of the above-mentioned cities, Dinges (1982) also investigated the early studies of WSP in USA. For instance in 1938, in Texas, there was an investigation into the biochemical functions that take place inside WSPs, which was reported by Peurifoy. He also found Hall studied the WSP's microbiology in 1936 in Texas. California, Nevada and Arizona were the states which he found were being proposed for the use of WSP system to treat discharged community wastewater as a result of a programme carried out by Caldwell in 1946.

Results from early studies into the capacity for effective sewage treatment by the WSP system led to the scientific acceptance of WSP and has been included as one conventional sewage treatment system since the 1940s (Sheikh 1981, Widmer 1981). Thus, the WSP system has been utilised for community wastewater treatment processes, in several parts of the world, in both the northern and southern hemisphere.

Even in the Arctic region, WSPs have been adopted for sewage treatment and have been reported to operate successfully despite extremely low temperatures (Walters 1961). Also in places located in very high altitudes such as Cajamarca (2675 m) and Juliaca (3827 m), cities in Peru, WSPs were said to perform with satisfactory results (Pearson 1987).

Countries reporting the use of ponds for wastewater treatment from 1967 include Argentina, Australia, Bolivia, Brazil, Colombia, Costa Rica, Cuba, Ecuador, Ghana, Guatemala, India, Israel, Kenya, Mauritius, Mexico, Nicaragua, Nigeria, Pakistan, Peru, Saudi Arabia, South Africa, Thailand, Uganda, the United Arab Republic, United States of America, Venezuela, Zambia and Zimbabwe (Gloyna 1971). Another list of countries adopting sewage ponds includes Barbados, Chile, the Dominican Republic, El Salvador, Honduras, Malaysia, Panama, Tanzania, Uruguay and Vietnam (Reid 1982). These countries have used a WSP system ranging widely from those serving small communities to large scale operations for sewage purification.

Several authors like Theodore (1956), Svore (1964), Marais (1970), McGarry and Pescod (1970), Gloyna(1971), Arceivala (1972), Thirumurthi (1974), Gloyna, Malina and Davis (1976), Mara (1976), Baumann and Karpe (1980), Arthur (1983) and Bradley (1983) have long been advocating the WSP process as an effective means of wastewater treatment system particularly in providing a very high yield of nutrient and organic removal from treated effluent.

Where rural communities, in various regions of the world, have developed their own sewage treatment systems, the WSP system was found to be the most commonly recommended sewage treatment process. This was mostly due to its low cost of construction and maintenance, good effluent quality and other reasons similar to those mentioned in Chapter 2.

A programme of the WSP system at a national level for sewage treatment could be found in countries such as India (Lakshminaravana 1972), United States of America (Towne and Horning 1961, Schurr 1970, Richmond 1970, Pierce 1974), England (Potten 1972), South Africa (Staler and Meiring 1965). Also, the WSP is claimed to be a feasible option for sewage treatment in developing countries, since it was found to be more effective in warm climates where sunlight is prevalent (Staler and Meiring 1965, Gloyna 1971, Ansari 1973, Pickford 1977, Reid 1982, Arthur 1983). Not only was it reported to produce good results in tropical and sub-tropical regions, but in colder climates as well such as in Germany (Bucksteeg 1987), Canada (Soniassy and Lemon 1986, Mathavan and Viraraghavan 1991), and Scandinavian countries like Norway (Odegaard, Balmer and Hanaeus 1987) or even Alaska (Clark 1970).

There have been at least five international symposia devoted to the WSP system. The first was held in Kansas City, Missouri, USA 1960, the second in Nagpur, India 1963. These two pioneer conferences were mainly involved with describing some experiences of countries using WSPs system. The third WSP seminar was organised in Kansas City, Missouri, USA 1970 and was largely aimed at exchanging designs of the WSP system. The fourth was at Lahore, Pakistan 1980 which was still focused on the topic of WSP designs and operations but topics discussed were mainly related to practices at regional levels especially in the Eastern Mediterranean region. The latest conference on WSPs was held in Lisbon, Portugal 1989 where major issues considered the application of the WSP system in treating agricultural and industrial wastewaters. Reviews of WSP performance in some countries in Europe, the Middle East, South Africa and South America were also made.

From the above conferences, some interesting recommendations were made for future WSP studies. These were: (1) the search for WSP design models consistent with actual results from WSP treated effluent, (2) the study of the final effluent of WSPs in terms of compliance with a country's effluent standard, (3) the investigation and upgrading of WSP performance, and (4) issues associated with the application of WSP effluent for land and crop irrigation. The current trend of studies involving WSP concerns areas of interest covering many facets of disciplines such as public health, engineering, environmental impact assessment, geology, biochemistry and microbiology.

By its simplicity, low initial capital investment and low energy input requirement, the WSP has become of major interest to city managers in developing countries (Gloyna 1971, Pickford 1977). The other main interest in developing countries is the issue of environmental health which is currently of major global interest. A concrete example is the worldwide campaign as a result of "The International Water Supply and Sanitation Decade 1981-1990" which was set up by the World Health Organisation (WHO) (Hess 1983). As a response to this campaign, financial assistance has been extended by the World Bank to develop wastewater schemes in developing countries. It has agreed to contribute financially to any national government which requires funding for wastewater treatment projects (Arthur 1983).

To revisit the preliminary development stage of WSP in general, the main concern was the issue of WSP's application to domestic sewage treatment. For instance, thirteen communities in North Dakota were found to use the WSP system as sewage treatment lagoons (Williamson 1956, Svore 1961). Later in the late 1960s, some WSP inventions for industrial wastewater treatment process were documented.

In Canada, the WSP system has been applied to treat wastewater from mining, meat packing, poultry processing, rendering, canning, potato processing, dairy products and even oil processing industries (Voege and Stanley 1963).

In Asia, published documents on WSP tend to emphasise the WSP's use as a natural reservoir for treating by-products as well as sewage, rather than its use as a conventional sewage treatment process. In addition, there are very few formal documents on the use of this type of sewage treatment system in Asian countries. So far, only India has carried out some research projects, conducted by the Central Public Health Engineering Research Institute on a WSP in Nagpur (Lakshminarayana 1972). The WSP mentioned was a large scale domestic sewage treatment system. Smaller scale WSP utilisation was reported by McGarry and Pescod (1970) who investigated WSPs for domestic wastewater treatment at the Asian Institute of Technology in Bangkok, Thailand. The latter, however, was more small scale than those implemented in the cities of Western countries.

WSP models suitable to hot climates have been studied by Arceivala (1970), McGarry and Pescod (1970), Gloyna (1971), and Mara (1976) and eventually they have proposed formulae designed for WSP systems in tropical countries. Other WSP studies in Asia merely surveyed numbers of WSP used for community sewage treatment in the locality, and did not provide any model or introduce a scientific approach.

However, in recent years after some countries received funding from the World Bank for constructing their WSPs, some studies aimed at an assessment of the WSP system to determine whether it was effective in treating sewage in large cities (Arthur 1983). Arthur (1983) noted, in the conclusion drawn from these WSP assessments, that there was a wide diversity of WSP designs adopted. Moreover, he found that sewage personnel still needed practical experience in designing, locating, constructing and operating WSP systems in order to achieve more effective performance in their communities.

It is important to note that even when the WSP is widely used in various areas, no two WSP operations are alike, each of them performing differently. These varied performances are seen as a result of different environmental and wastewater characteristics of the localities (Dinges 1982). Therefore, an investigation of WSP's performance as well as its design and operation is still required. The outcome from that investigation will be, to a large extent, beneficial to the locals in developing WSPs.

In general practice, WSP is used as a unit in both secondary and tertiary sewage treatment processes. WSP is used for the secondary treatment process, an example of which is the WSP plant in Altamira City, Mexico (De la O and Martinez 1976). One example of WSP for the tertiary treatment process in domestic sewage treatment is in Gumeracha City, Adelaide, Australia (Mitchell and Williams 1982). The use of WSP for industrial wastewater treatment plant, for example, one of the largest WSP plant using WSP as a tertiary wastewater treatment unit is exemplified by the Union Carbide Corporation Plant, Texas, USA (Martin *et al.* 1976). In Australia, there are documents which show that WSPs have been widely used for secondary and tertiary treatment. There are approximately 300 communities throughout Australia using a WSP system serving populations of around 100 to over 100,000 per plant (Mitchell and Williams 1982).

3.2 Definition of terms

WSPs are large shallow basins enclosed by earthen embankments in which raw sewage is treated by entirely natural processes involving both algae and bacteria (Mara 1976). These natural biological processes are generally aimed at the *reduction of organic contents* (mainly carbonaceous BOD_5) and *destruction of pathogenic organisms* in wastewater discharged from communities (Marais 1970, Gloyna 1971, Mitchell and Williams 1982). With respect to domestic wastewater, the major objective of WSPs is to reduce the organic content, and in many cases, nutrients such as nitrogen and phosphorus (Tchobanoglous 1991).

The term "Waste Stabilisation Ponds" (WSPs) has been used by organisations along with "wastewater lagoons", "waste stabilisation lagoons", "oxidation ponds" and "sewage lagoons". These terms are also used to describe any pond designed as a biological wastewater treatment system (Middlebrooks *et al.* 1979). Gloyna (1971) first attempted to organise terminology by concentrating on definitions in association with biological conditions occurring within the ponds. These terms are:

(1) *Anaerobic waste stabilisation pond* is the pond which is designed to maintain anaerobic conditions throughout the entire water column. Anaerobic conditions occur in this pond as the rate of oxygen consumption exceeds the reaeration rate.

(2) *Aerobic waste stabilisation pond* refers to the pond employing bacteria which requires oxygen to break down organic wastes, a term which also specifies an oxygen source coming from natural mechanisms including, both photosynthesis by algae and reaeration by the wind.

(3) *Facultative stabilisation pond* is the pond where three layers of aerobic, intermediate and anaerobic zones are expected to occur. The upper layer is aerobic where aerobic bacteria and algae inhabit symbiotically.

The middle layer is partly anaerobic and partly aerobic, where facultative bacteria degrade organic wastes. The bottom zone is anaerobic, where anaerobic bacteria degrade settled solids.

(4) *Aerated waste stabilisation pond* is a pond which has a mechanical aerator for supplying oxygen to the pond.

The above terms are mainly defined according to oxygenation conditions in reaction to bacteria existing within the pond. However, in practice, the oxygen present in any pond is largely dependent on the strength of loadings. When a high organic load flows into the pond system, anaerobic conditions can be expected in all of the ponds where the oxygen consumed exceeds reaeration rate. In contrast, if the pond receives a low organic loading, aerobic conditions prevail throughout the pond system. Other than organic loading, physical and chemical factors can also affect the oxygen presence in the pond. So, the terms used from (1)-(4) above largely explain the general concept of pond design displaying certain features associated with oxygen presence in the WSP system.

When considering the treatment process, WSPs are classified as follows (Widmer 1981):

(1) *Primary waste stabilisation pond* refers to the first pond receiving raw sewage and this term is used whenever there are one or two ponds in a series connected to this first pond.

(2) *Secondary waste stabilisation pond* is the term for the pond receiving effluent from the primary waste stabilisation pond.

(3) *Tertiary waste stabilisation pond* is the third or subsequent pond in a series receiving effluent that is usually well stabilised. This pond is also termed a *maturation pond* which is defined as a tertiary pond having the function of reducing organisms harmful to health, and it is also known as a polishing pond or fish pond. This type of pond is normally preceded by primary and secondary waste stabilisation ponds.

The utilisation of the various terms may be exemplified by current research being carried out by several investigators. For instance, those whose studies focus on biochemical reaction within ponds, the terms facultative and maturation ponds are usually used in order to reflect such mechanisms. Whereas, others who might be interested in the engineering aspect of a pond system, use the second group of terms i.e. primary, secondary and tertiary ponds. The varying use of terms occurs because the latter group is interested in pond processes, while the first group focuses on biochemical science. Nevertheless, the two groups of terms are often used interchangeably among researchers depending on their personal purpose.

Purposely in this study, the terms facultative and maturation ponds will be used in order to reflect the biochemical mechanism inside the ponds and also the ponds' performance and effectiveness as domestic sewage pond treatment systems.

3.3 Structure of WSP systems

WSPs are usually arranged in a series or parallel. More recently, the structure tends to follow a series pattern for the reason that multi-cell ponds tend to perform better in providing good effluent quality compared to one pond of equal volume (Gloyna and Herman 1956, Metzler and Culp 1959, Parker 1962, Oswald 1963, Marais 1974, Mara 1975, Bartone and Arlosoroff 1987, Santos and Oliveira 1987). However, there are some WSPs arranged in a series and have been found to display poor performance e.g. in Jordan (Salem and Lumbers 1987). The multi-cell approach and the rectangular shape of WSPs in general is aimed to prevent short-circuiting from large amounts of inlet organic wastes and also provides for more flexible operation of WSPs.

The application of conventional WSP systems can usually be classified into two groups, the first is WSPs for secondary treatment, and the second is their use as a tertiary unit. For the first group, WSPs are normally preceded by a primary or pretreatment unit. This comprises a screen and grit chamber for removing floating mats of detritus from raw sewage such as found in the Khon Kaen WSP in Thailand (TISTR 1986b). WSPs operating as a tertiary unit are often arranged in a line like conventional secondary wastewater treatment units such as activated sludge, Imhoff tank or trickling filter. An example of this is in the rural areas of Germany where WSPs are preceded by trickling filters (Bucksteeg 1987), and also in the rural town of Gumeracha, Adelaide, Australia (Mitchell and Williams 1982).

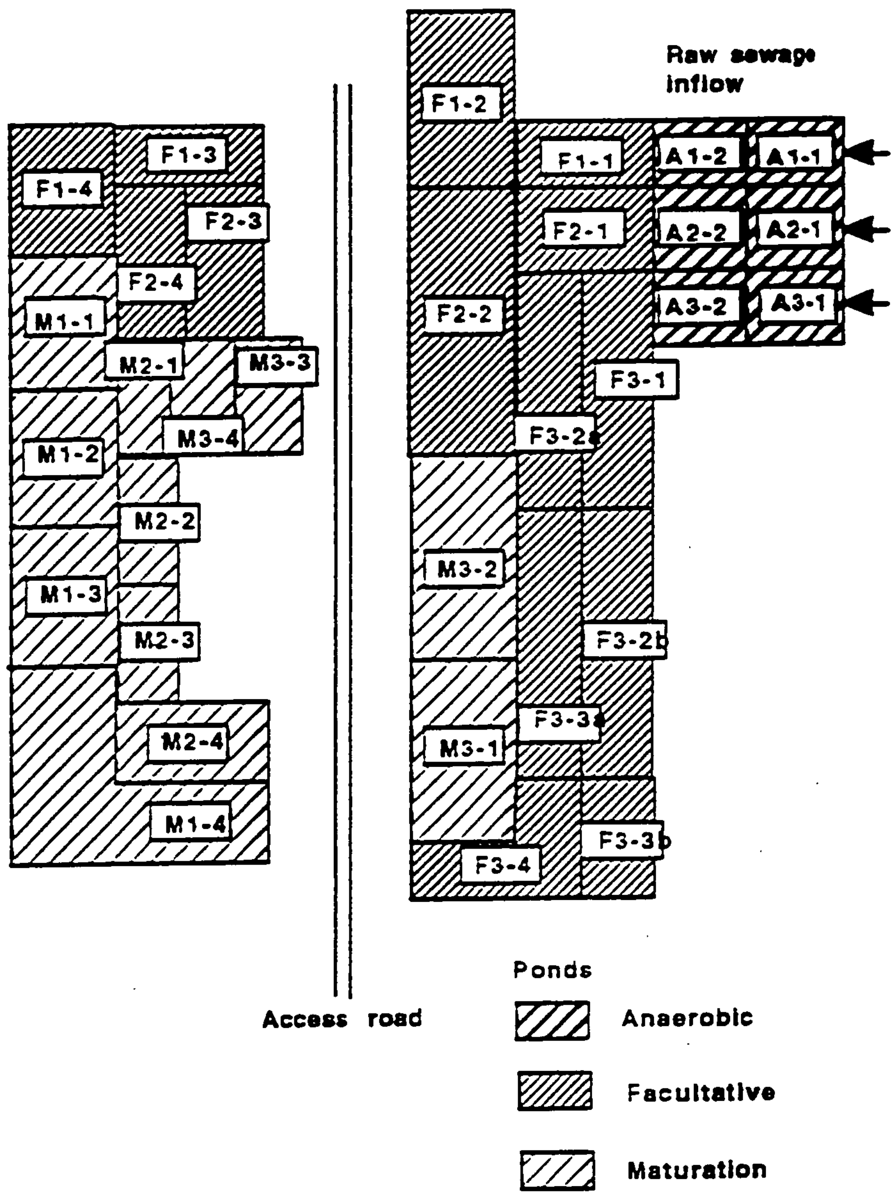
The figures below show the usual features of several WSP arrangements. These are: (a) a wastewater treatment system with facultative and maturation ponds where the last unit of maturation pond is optional, (b) when the facultative and maturation ponds are preceded by an anaerobic pond, in which this type of pond also theoretically includes the WSP concept, (c) a WSP used for tertiary treatment which is normally known as a polishing pond, (d) a WSP acting as a biological wastewater treatment digester like in (a) but followed by a disinfection unit, and (e) is the scheme of a WSP system combined with an aquaculture pond for fish rearing.

(a) screen and grit chamber-anaerobic-facultative-maturation ponds
(b) screen and grit chamber-facultative-maturation ponds
(c) screen and grit chamber-secondary treatment unit-WSPs
(d) screen and grit chamber-WSPs-disinfection
(e) screen and grit chamber-WSPs-aquaculture ponds

The above common arrangements of WSPs were derived from McGarry and Pescod (1970), Gloyna (1971), Tam (1982), Ellis (1983) and Hess (1983). The major concepts were obtained as follows: for (a) it is usually applied to plant with a very highly polluted raw wastewater which requires an anaerobic pond to reduce impurities to a certain level before discharging wastewater into facultative and maturation ponds. For (b) the WSP's entire role is to purify sewage, whereas in (c) WSPs are employed to treat effluent from a conventional unit which is viewed to be incapable of purifying sewage effectively. WSPs in (d) include an additional disinfection unit for WSP systems which cannot cope with the elimination of disease-causing bacteria. And finally, WSPs in (e) are extensively adopted in aquaculture for the purpose of reducing algal content in the final effluent, which fish or other aquatic animals feed on as a protein source.

Figures 3.1, 3.2, 3.3 and 3.4 show some examples of schematic layouts of existing WSPs which are being used for wastewater treatment. Figure 3.1 illustrates an arrangement of WSPs constructed in Jordan in 1985, receiving volumes of 57,000 m^3 of sewage per day. It is a series of ponds constructed in three parallel lines. Each line comprises two anaerobic ponds, four facultative and four maturation ponds (Salem and Lumbers 1987). Figure 3.2 shows the typical layout of a series of ponds which were constructed in line formation from the anaerobic pond through to maturation ponds (Lansdell 1987). Figure 3.3 shows a series of tertiary WSPs preceded by a secondary treatment unit, that of trickling filter. Both ponds were built in 1965 with the same depth of 1.1 m (Mitchell and Williams 1982). Figure 3.4 presents a small scale WSP in Ardi Institute, Tanzania which was constructed in series and ended up with a fish pond. The first FP, second FP, the MP and the fish pond of this WSP have depths of 1.43, 1.33, 0.85 and 0.8 m respectively (Yhdego 1992). Arthur (1983) evaluated WSPs in six countries: Philippines, India, Israel, Kenya, Zambia and Jamaica, all of which were funded by The World Bank. His proposed layout plan for WSPs, resulting from the evaluation, was then published and is shown in Figure 3.5.

Figure 3.1 The layout plan of Al Sumra WSP plant in Jordan.
(Source: Salem and Lumbers 1987)



Figures 3.2, 3.3 and 3.4 Schematics of WSPs in Venezuela (Figure 3.2, Landsdell 1987), Australia (Figure 3.3, Mitchell and Williams 1982) and Tanzania (Figure 3.4, Yhdego 1992).

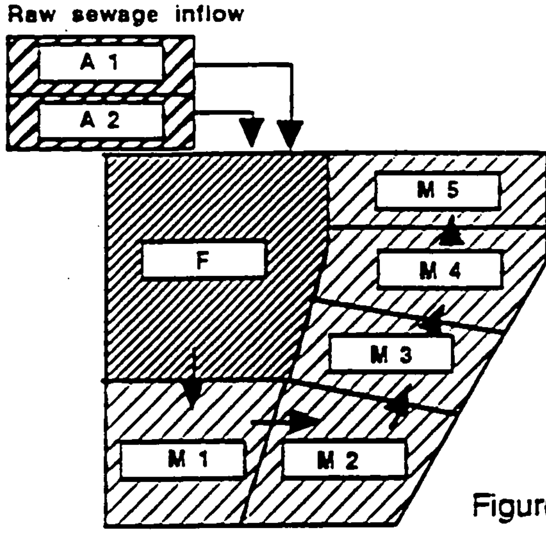


Figure 3.2

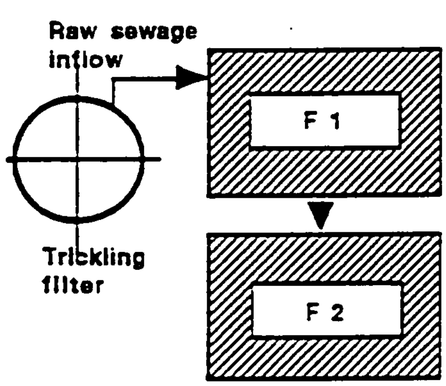


Figure 3.3

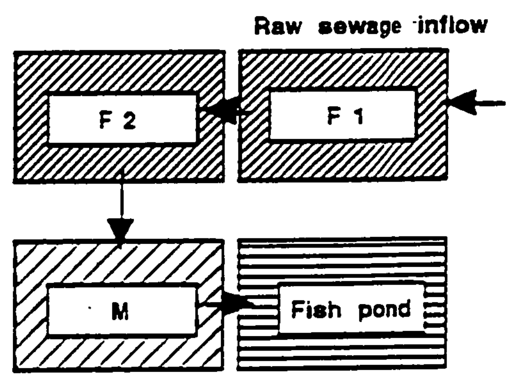


Figure 3.4

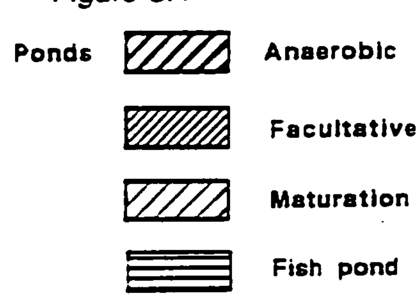
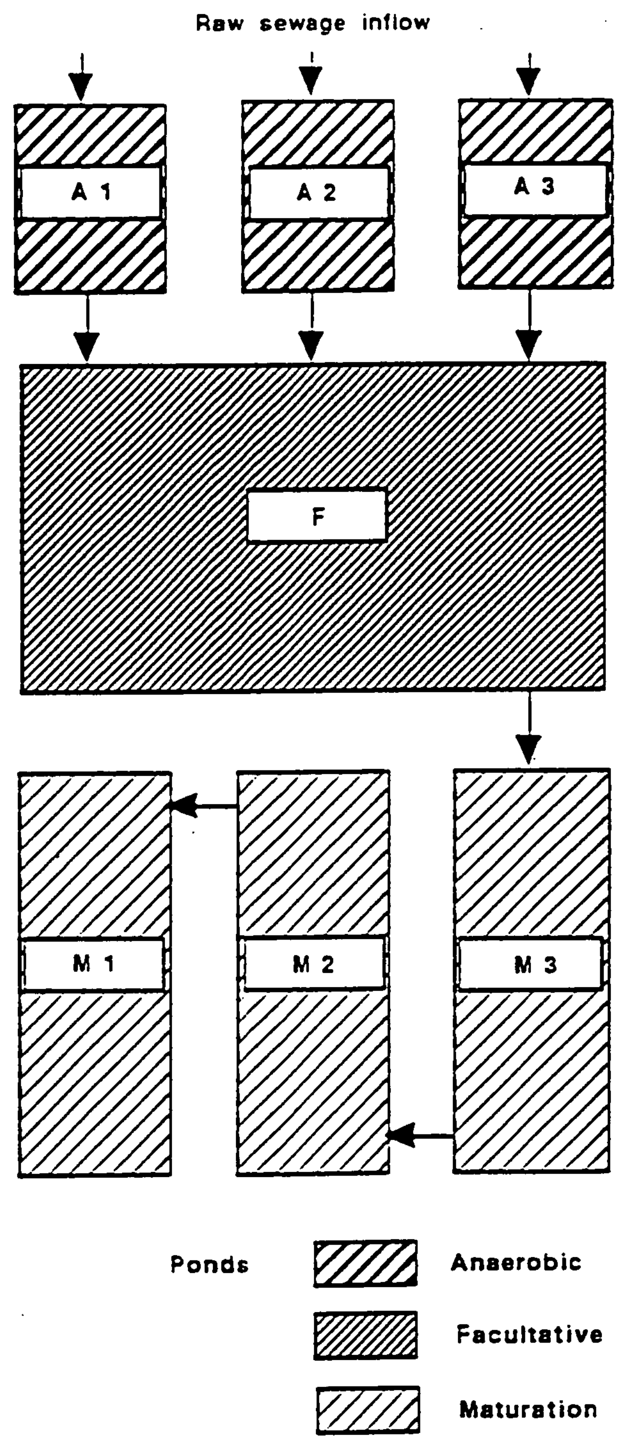


Figure 3.5 The schematics of the WSP system proposed by Arthur (1983).



The shape of ponds within WSP systems is usually square or rectangular (Hess 1983). However, Mara (1976) reported that according to hydraulic characteristics, rectangular ponds performed better than square and circular ponds. A length to breadth ratio of ponds of 2 to 1 and 3 to 1 is common (Mara 1976). Dinges (1982), based on experience in the USA, recommended that the ratio of length to breadth should be at least 5 to 1, shaped as a narrow rectangle. The purpose of this is to take advantage of the prevailing winds which help reaerate the pond. Secondly, it is expected that average flow velocities are higher, increasing scouring effects, minimising dead space and helping to promote reactions like denitrification that occur predominantly in pond sediments. Lastly, an increased length to breadth ratio can cause the flow pattern to be closer to a plug flow type, and this should assist the attainment of lower effluent concentrations of, for example, BOD.

For the ponds' shape, Middlebrooks *et al.* (1979) in a literature review found that there was no agreement among investigators about whether a pond should be rectangular, square or circular for more effective treatment. However, the rectangular design is more commonly concerned with preventing short-circuiting of sewage flow. Hess (1983) noted that the slope of the pond should vary according to the type of soil.

Still, there is no one specification for pond configuration. It is largely dependent on preferred design criteria; land topography suitability; or the limitation of local conditions. However, some current practices seem to favour a rectangular shape with a ratio of length to width of 2 to 1 (Baumann and Karpe 1980). In summary, in considering the relation of pond shape to the capability of treating sewage in developing countries, documents reveal that regardless of shape, ponds are performing well, for example the use of a square type in Nagpur City, India (Lakshmanarayana 1972) and rectangular in Dar es Salaam City, Tanzania using a rectangular pond (Yhdego 1992).

Depth of the pond is another factor which appears to vary. However, some renowned researchers of WSP systems, like Oswald *et al.* (1964), noted that for the most efficient operation of WSPs, the facultative pond (FP) should be designed to operate at a depth of between 1.5 and 2.0 m. Mara (1976) documented that FP should have a depth of 1 to 1.5 m and maturation pond (MP) depth should be the same. For anaerobic ponds (AN), the depth should be 2 to 4 m. Another World Bank researcher, Arthur (1983), recommended that FPs should be 1.8 m whereas MPs should be similar to that depth suggested by Mara. For AN, it was suggested that 4 m is favourable. In general however, WSPs are mostly recommended to have a depth ranging between 1 and 1.5 m (Brinck 1961, Fleming 1962, Mara 1976, Pescod and Mara 1988).

It should be noted that the variety of depths of WSP systems, as documented by several investigators, occurs because of the difference in organic loading of a given community. A shallower pond aims to increase oxygen presence. In any community which contributes a lower load, a deeper pond tends to be recommended since the expectation is that oxygen consumed is not greater than the rate of reaeration. Secondly, environmental factors such as temperature, sunlight, biota existing the ponds and so on vary from one community to another. The second criterion also has to be taken into account in conjunction with the first loading criterion and these combine to result in the different WSP depth recommendations.

Table 3.1 shows some countries where WSPs have been constructed. The types and depths of ponds are given and their arrangement in series and/or parallel sequence is also shown.

Table 3.1. The type, arrangement and depth of WSPs in some countries.

Location	Types and arrangement	Depth (m)	References
Latin America			
Canas, Costa Rica	FP parallel	0.5	Saenz (1969)
Lima, Peru	FP parallel	0.3	Talboys (1971)
Brasilia, Brazil	AN-FP series	0.5	Talboys (1971)
Canal Zone, Panama	AN-FP series	1.0	Eckley, Canter and Reid (1974)
Palmira, Columbia	FP series and parallel	1.0	Canter (1969)
Asia			
Madras, India	AN-FP series	1.5	Purushothaman (1970)
Nagpur, India	FP series	0.5	Lakshminarayana (1972)
Nagpur, India	FP parallel	0.5	Lakshminarayana *et al.* (1969)
Malaysia	FP series	0.5	Bradley (1983)
Africa			
Mandarellas, Zimbabwe	FP series	1.8	Hodgson (1964)
Nairobi, Kenya	FP series	0.5	WHO (1973)
Dar es Salaam, Tanzania	FP-MP-Fish Pond	0.8- 1.5	Yhdego (1992)
Australia			
Gumeracha, Adelaide	TF-FP series	1.0	Mitchell and Williams (1982)

A variety of longitudinal-section designs for WSPs have been proposed by many authors. Mara (1976) noted that to enhance both aerobic and anaerobic conditions inside FPs, a typical layout as illustrated in Figure 3.6 is recommended. Cross-section and interpond connection structures, as in Figures 3.7 and 3.8 were proposed by Mara (Mara 1976).

Figure 3.6 A longitudinal dimension of the facultative pond by Mara (1976).

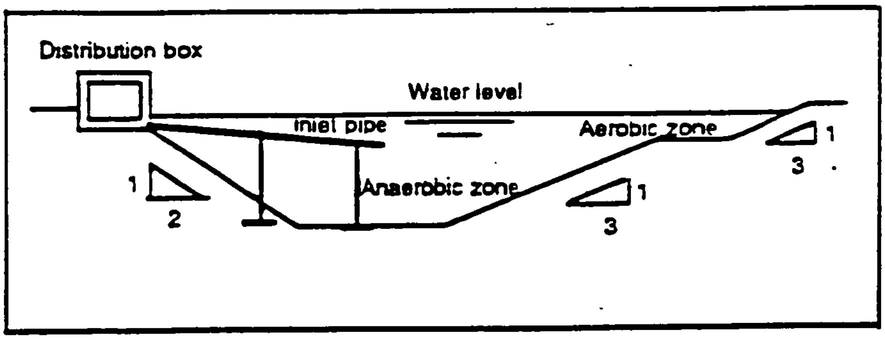


Figure 3.7 A cross-section dimension of the facultative pond by Mara (1976).

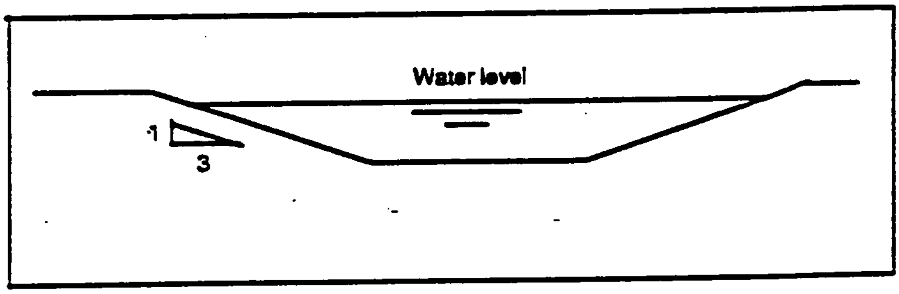
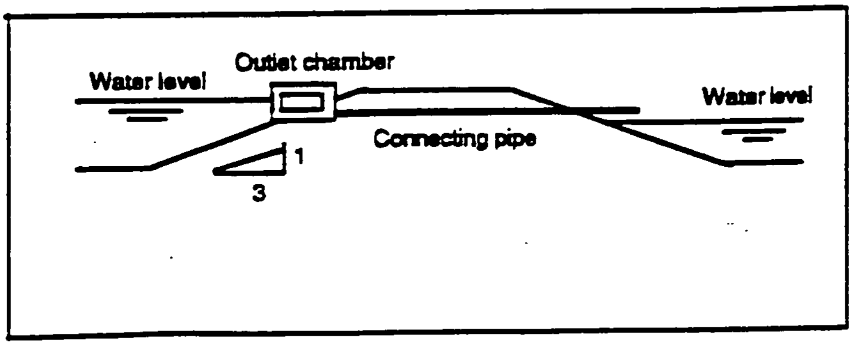
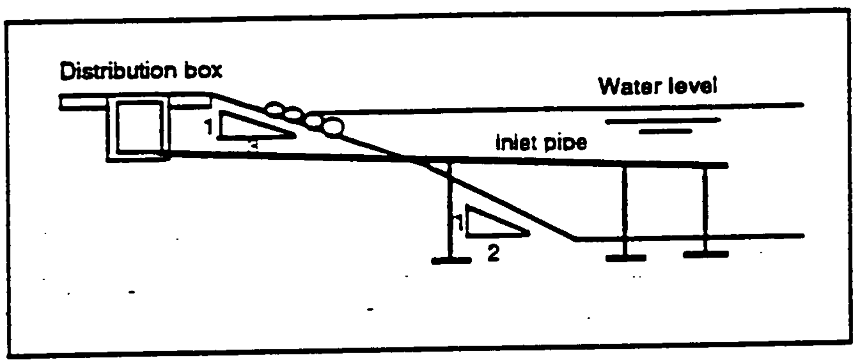


Figure 3.8 A connection plan between WSPs via outlet chamber and pipe by Mara (1976).



Arthur (1983) also documented an FP longitudinal-section and interpond connection design as illustrated in Figure 3.9. This is similar to that of Mara (1976) but the FP design by Arthur did not concentrate so much on dividing aerobic and anaerobic zones.

Figure 3.9 The structure of a facultative pond by Arthur (1983).



The above longitudinal dimension of the pond is presented in order to show the inlet structure which is viewed as significant for preventing short-circuit of sewage flow in the pond. In addition, it supports a mixing of fluid which would result in initiating mineralisation within the ponds. For the cross section design, different layouts were proposed by some authors in the contexts of slope and depth of the ponds. For example, Tam (1982) proposed a cross-section plan of a pond as shown in Figure 3.10. This cross-section plan gave a liquid depth of 3 m with inside slope 1 to 2.5, outside slope 1 to 1.5 and free-board of 0.5 m (the distance between the dyke level and water level of the pond). Similarly with regards to pond dimension, Hess (1983) suggested that liquid depth should be 2 m with inside slope 1 to 3, outside 1 to 1.5 and free-board of 0.5 m. However, Hess also shared Tam's suggestion of dyke's width being 2 m. This is shown in Figure 3.11.

Figure 3.10 A cross-section plan of the waste stabilisation pond by Tam (1982)

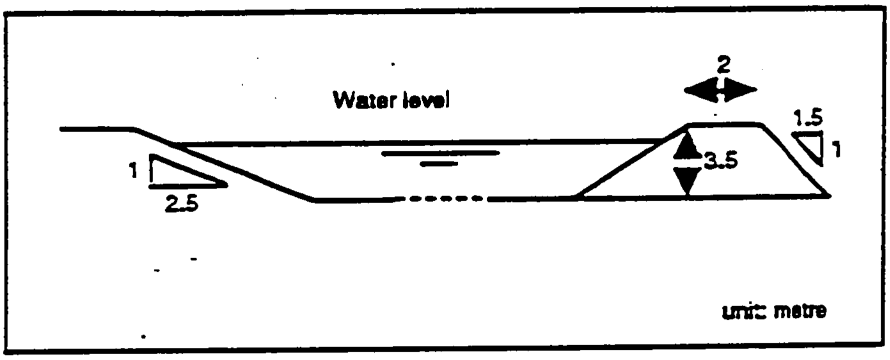
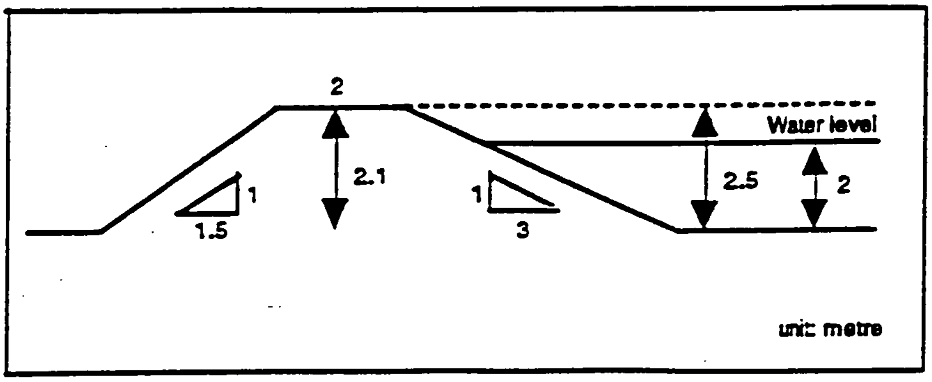


Figure 3.11 A dimension of the waste stabilisation pond designed by Hess (1983).



Considering depth, Middlebrooks *et al.* (1979) noted that depth is the most important factor in WSPs. The effect of increased depth will result in a condition of decreased light intensity penetrating the liquid zone. The main concern is that of the aerobic condition of the pond. This was initially emphasised by Oswald *et al.* (1964). To facilitate the pond's aerobic function throughout its depth, he suggested that the pond should not be more than 0.30 m deep. This was different from that argued by Middlebrooks *et al.* (1979). Middlebrooks viewed that when the pond depth is equal or less than 0.30 m, duration of pond usage might be less. This occurs due to the sludge which would rapidly accumulate from the bottom to the upper layer of the pond column. Further, Ellis (1983) documented that such a shallow pond would enhance the growth of rooted weeds and filamentous algae. These would take root on the bottom and as a result disrupt flow, trap detritus, and inhibit mixing. In addition, Mara (1982) revealed that depths below 1 m could not prevent vegetation growing up from the pond and depths of more than 1.5 m would make the pond predominantly anaerobic rather than aerobic.

In considering how pond depth could effectively control emergent vegetation along the embankment of the pond, Middlebrooks *et al.* (1979) recommended that this depth would be 1 m. In order to prevent emergent plants from invading the pond, Dinges (1982) noted that a depth of 0.9 m is becoming the standard depth of ponds in the southwestern United States. But this depth increases in the Midwest region up to 1.5 m.

To a large extent, determining the optimum depth of WSPs is dependent upon geographical and geological conditions. Evidence from the USA shows there has been an establishment of design criteria for WSPs in American cities, mainly sponsored by the US. Public Health Service. The achieved results show a vast variety of pond depths in current use. These depths are shown in Table 3.2 below.

Table 3.2. Application of WSP depths in the United States of America.

Locations	Depths (m)	References
South Dakota	1.0-1.5	Allum 1955
Oklahoma	0.7-1.5	Assenzo and Reid 1966
Washington	0.7-1.2	Berschauer 1961
Montana	1.5	Brinck 1961
Nevada, Arizona	1.0	Caldwell 1946
Tennessee	1.0-1.5	Fleming 1962
California	1.5-1.8	Oswald 1961
Illinois	1.8-2.5	Rogers 1961

The preceding WSP depths are based on local investigations and experiences in the USA. To be more specific on the biological reaction approach, as in an FP which requires both aerobic and anaerobic conditions within the pond, it was first suggested by Marais and Shaw (1961) that both FP and MP ponds should have a depth of 0.9 to 1.5 m. The current practice, accepted by some authors, is that FPs' depth should be between 1.5 and 2.0 m (Brinck 1961, Fleming 1962, Mara 1976). Also Ellis (1983) noted that for FPs, the effective depth should be 1.5 m. Widmer (1981) documented that FP depth in hot climates should be at least 1.5 m and best at a 2 m depth. With respect to MP depth, Mara (1982) and Widmer (1981) suggested that MP depth should be 1 - 1.5 m.

However, in some WSP plants, there are still practices different from the recommended MP depth. An example is that of a WSP in Dar es Salaam, Tanzania where the depth of 0.85 m was implemented (Yhdego 1992).

It seems that pond depth still requires further investigation and cannot be specified as a certain value. It might be that the pond depth has to be adjusted to suit local conditions which vary greatly. Furthermore, with increasing land costs, a deeper pond would be viewed as preferable to a shallower one in terms of increased volume capacity for treating sewage, especially in cities with a population exceeding 100,000. The current trend is moving towards deeper ponds in a series configuration. The most recent example was the investigation of WSP system performance with 2.2 deep m ponds by Silva, Mara and Oliveira (1987).

Another important feature of ponds is the embankment. As mentioned earlier, this structure is usually made with varied slope but normally of 1 to 2 - 1 to 3 (Mara 1982). The usual physical character of an embankment is one of pre-cast concrete slabs or stone lining the slope. This is to protect embankments from wave erosion caused by wind-induced waves within the pond. Also, this structure prevents vegetation growing down the embankment and creating an ideal habitat for mosquitoes. The top of the dyke as shown in Figure 11, is usually made of compact clay, or lined with asphaltic concrete to provide a support for operators maintaining the ponds and for traffic. Hess (1983) recommended that the width of this should be no less than 2.5 m.

Pond base is another important consideration of the WSP structure since it affects wastewater infiltration into the deeper soil layer or underground water. Compact clay is favourable for lining the pond base which Hess (1983) suggested to be 10 m thick. Polyethylene sheet, puddled clay, bitumen or asphalt have also been used (Mara 1982) for this purpose.

However, according to Hess (1983) the use of plastic sheets often results in a high cost of construction in developing countries. Another alternative, as Hess (1983) recommended, is that of using soil-cement to seal the pond base which has provided a good performance such as in Brazilian farms.

To a large extent, the choice of pond structure is dependent on many factors. The arrangement of the pond system either in a series or parallel is aimed mainly at preventing short-circuiting and to assist the pond system in achieving a higher quality effluent. The number of ponds in any arrangement is determined by the degree of BOD strength. Wherever there is a high BOD content, a large number of ponds in a series or parallel will be used.

The pond dimension concentrates on shape and depth. The ratio of the length and breadth determines the degree of mixing in the pond. A greater ratio of length to breadth is expected to enhance better mixing inside the pond. Such a ratio will generate what is known as the plug flow pattern which provides the pond with improved biodegradable reaction of organic matter to microorganisms and also assists algal culture to take up nutrients.

The pond depth is seen to be the crucial factor which will determine the biochemical and chemical functions inside the pond. With an increase of pond depth, anaerobic conditions can be expected at the bottom zone of the pond. Sunlight cannot penetrate to this depth since photosynthesis by algae does not occur in this layer and oxygen presence cannot be expected. pH in this layer is normally found at a low level. This acidity condition results from the production of acids when anaerobic bacteria degrades settled solids.

However, this lower pH and the absence of oxygen can enhance denitrification reaction which finally releases nitrogen gas. The end products of a deeper pond, which is anaerobic, are mostly the gases CH_4, H_2S, N_2 and CO_2. Acid-forming and methane-forming bacteria have a dominant role in this.

For the shallower pond, a greater amount of oxygen can be expected and organic matter is oxidised by various aerobic bacteria species, CO_2 produced from such biooxidation will supply to algal photosynthesis. Sunlight can also penetrate throughout the shallower pond and thus can activate the algal photosynthesis.

As concluded above, the pond depth will condition the biochemical and chemical activities inside the pond but this must be considered in conjunction with another factor which determines the pond depth, the organic strength. The high organic loading normally needs a deeper pond which is capable of treating a higher load whereas the shallower one can handle a lower load. So, the determination of pond depth in any WSP system will be dependent on the nature of the community sewage.

3.4 Pond design

It is necessary to review the current practices of WSP design since this will consequently affect a pond's function. In this review, the approach will be extended only to the most commonly used design models involving facultative and maturation ponds. Further, according to some of these formulae, the studied WSPs will be tested for results, to determine whether or not the results are in agreement with the formulae used.

In general, Gloyna (1968, 1971) concluded that any pond design applied will be more effective if criteria used include local environmental considerations and experience from each region. Reid (1982) summarised that principally, WSPs are designed on the basis of organic loading, depth, and detention time of the fluid in the ponds. Design approaches other than presented by Reid (1982) were mentioned in Gloyna (1971). In that document, WSP designs are determined by factors of temperature, light, volumetric loading, bottom sediment accumulation, toxicity of waste, size and shape of pond facilities. For conventional WSP design formulae, Mara (1976) and Middlebrooks *et al.* (1979) have agreed that the development of design criteria is mainly based on a combination of experiences and empirical equations.

For facultative ponds, there are many current design procedures which have been introduced to WSP construction, for instance, formulae of Marais and Shaw (1961), Arceivala (1970), McGarry and Pescod (1970), Gloyna (1971), Mara (1976), Yanez (1980) and the Asian Institute of Technology (1981). These design equations are created from different bases. For example, Mara's equation is mainly based on hydraulic retention time (HRT), Marais and Shaw's formulae concentrated on depths and HRT, Gloyna's model focused on temperature which is similar to McGarry and Pescod's, whereas formulae of Arceivala centred on location of WSPs.

However, it is interesting to note that some of the commonly used design formulae e.g. those of Marais and Shaw (1961) and Mara (1976) have developed models following the first order kinetic biooxidation concept. This first order kinetic is clarified as the rate at which organic matter is oxidised by microorganisms in ponds and is a function of time. In other words, the rate of BOD_5 removal (rate of oxidation of organic matter) at any time is proportional to BOD_5 (organic matter) present in the system at that time. This biooxidation kinetic has been expressed in mathematical terms by Mara (1976), and is as follows:

$$\frac{dL}{dt} = -kL \qquad \text{Equation 1}$$

where L = amount of BOD_5 remaining (organic materials to be oxidised) at time t

k = first order rate constant for BOD_5 removal (d^{-1})

Equation 1 is used to explain the bioreaction occurring in WSPs which means that the rate at which the organic matter is oxidised is the differential coefficient dL/dt. The minus sign indicates a decrease in the value of L (organic matters or BOD) with time. The value of k is derived from Arrhenius equation where values of k are dependent upon temperature. This equation is as follows (Ellis 1982):

$$k_T = k_{20}\, Q^{T-20} \qquad \text{Equation 2}$$

where k_T and k_{20} = are the values of k at T ^{0}C and 20 ^{0}C

Q = is an Arrhenius constant (value of WSPs is 1.05)

From those two equations, Mara (1976) has applied and developed a formula for designing WSPs as follows:

$$\frac{L_e}{L_i} = \frac{1}{1 + k_T t^*} \qquad \text{Equation 3}$$

where L_e and L_i = are the values of BOD_5 (mg/L) for effluent and influent

t^* = is the mean retention time

k_T = is first order rate of constant BOD_5 removal at T ^{0}C (for WSP $k_T = 0.23\ d^{-1}$)

However, this formula at a certain point has a limitation since the first order kinetics model is largely based on an assumption that organic wastes are oxidised at the same rate and remain constant in amount over a given time. Later investigation revealed that in WSPs this rate will decrease whenever retention time increases, as was examined by Middlebrooks *et al.* (1979).

The second approach to the design of WSP formulae is that of Marais and Shaw (1961). This concept is still a modification of the first order kinetic, and additionally focuses on WSPs' depth influence. This formula is as follows:

$$L_p = \frac{600}{(0.18\,d + 8)} \qquad \text{Equation 4}$$

where L_p = Effluent BOD_5 (mg/L)
d = Pond depth

and further if the influent BOD_5 and detention time are known, design equation of WSP is then modified to a new equation below:

$$L_p = \frac{L_a}{0.17\,R_T + 1} \qquad \text{Equation 5}$$

where L_a = Influent BOD_5 (mg/L)
R_T = Detention time at temperature T

Another conventional WSP design formula is that of Gloyna (1971). This formula concentrates more on the effect of temperature and wastewater produced per capita. His equation is:

$$V = (3.5*10^{-5})\, Nq\, L_a\, Q^{(35-Tm)} \qquad \text{Equation 6}$$

where V = Pond volume (m^3)
N = Number of people contributing waste
q = Per capita waste contribution (litres/day)
Q = Temperature reaction coefficient = 0.085
Tm = Average water temperature of coldest month
L_a = Influent ultimate BOD_5 (mg/L)

Another design model was suggested by McGarry and Pescod (1970) who proposed two equations. One of the formulae is currently being used by several designers. The first equation is:

$$L_m = 11.2\,(1.054)^T \qquad \text{Equation 7}$$

where L_m = Maximum BOD_5 loading (kg/ ha d)
T = Ambient mean monthly temperature ^0F

The second equation of McGarry and Pescod (1970) for WSP is designed particularly for tropical Asia. This equation is as follows:

$$L_r = 10.75 + 0.725L_a \qquad \text{Equation 8}$$

where L_r = BOD_5 removal (kg/ha d)

L_a = Applied BOD_5 loading (kg/ha d)

However, from equation 7, Mara (1976) has modified McGarry and Pescod equation into a new design as shown in equation 9. Mara (1976) reasoned that this was because a safety factor should be taken into consideration. This equation will provide a much lesser value of BOD_5 than equation 7.

$$L_a = 7.5\ (1.054)^T \qquad \text{Equation 9}$$

where L_a = Design BOD_5 loading (kg/ha d)

Also, Mara (1976) offered another alternative design equation of WSP modified from equation 9 which he intends to present in a simple form. This equation is indicated as follows:

$$L_a = 20T - 120 \qquad \text{Equation 10}$$

where L_a = design BOD_5 loading (kg/ha d)

T = mean local temperature of the coldest month, ^0C, valid for $15\ ^0C < T < 30\ ^0C$

Yanez (1980), after his investigation of WSPs in Peru, modified the equation of McGarry and Pescod (1970), equation 8, to be a new equation for WSP as illustrated in equation 11:

$$L_r = 0.8193\ L_a - 7.81 \qquad \text{Equation 11}$$

where L_r = BOD_5 removal (kg/ha d)

L_a = applied BOD_5 loading (kg/ha d)

Arceivala (1970) also proposed a WSP formula which is mainly associated with latitude. This formula is:

$$L_a = 375 - 6.25L \qquad \text{Equation 12}$$

where L_a = design BOD_5 loading (kg/ha d)

L = latitudes are valid between 8^0 and 36^0

The Asian Institute of Technology (1981) also created a WSP design model. Its equation is shown as follows:

$$L_a = 8 * (1.054)^T \qquad \text{Equation 13}$$

where L_a = permissible load of BOD_5 (kg/ha d)
T = temperature ^0F

Arthur (1983) modified Mara's model (1976) and his WSP design equation is:

$$L_a = 20T - 60 \qquad \text{Equation 14}$$

where L_a = design BOD_5 loading (kg/ha d)
T = mean local temperature of the coldest month, ^0C, valid for 15 ^0C $<T<$ 30 ^0C

The above fourteen equations are normally used by designers according to different variables. As Middlebrooks *et al.* (1979) noted, although many investigators have created design models for WSPs, there were very few who had performed tests in terms of actual year-round data under different climatic conditions. Further, Ellis (1983) reasoned that design equations seemed to be established from experience rather than on a sound rational foundation.

Consequently, there is research being carried out in different parts of the world to test the efficiency of design formulae in compliance with effluent standard, modification of former models, or even the creation of new equations to suit that locality. All above-mentioned FP design equations are viewed to have a prime role in reducing organic content in liquid wastes. In other words, for decreasing BOD_5 in wastewater. In this study, some of those design models are intended to be tested regardless of whether they consistently fit to the expected results. Chapters 4 and 5 will present details about the test.

Though there are many empirical design formulae of WSPs contributed by researchers, other approaches are obtained from experience and data resulting from observed performance of historically existing WSPs.

One of these alternative approaches recommends a design for surface organic loading of WSPs derived from local experience and was later suggested to be at an optimal value of BOD_5 per unit area per day (generally termed as kg BOD_5 per acre per day, kg BOD_5 per hectare per day or pounds BOD_5 per acre per day).

The other value gained is that of per capita loading which was contributed by experience of an optimal number of per capita loading per unit area of pond. It is generally expressed as an optimal number of persons generating sewage per unit area of WSP (normally termed as number of persons per acre).

Both measures of organic loading have been widely applied to WSP construction, particularly FPs, in many countries. Normally, there are two main considerations of those actual organic loading values: firstly, these values can provide designers or environmental planners with confidence, in terms of outlet from sewage plant meeting effluent standards, and secondly such values can be used to compare values derived from contributed formulae to see whether or not they fit closely together.

One approach derived from local experience was the recommended organic loading values as exemplified in the following experiences from USA: Kansas City, 25-50 kg BOD_5/ha/day (Pierce 1960) and 50 kg BOD_5/ha/day suggested by Metzler (1959), 250 kg BOD_5/ha/day for northern Florida (Mills 1961), Minnesota 25 kg BOD_5/ha/day (Floan 1961, Rogers 1961), South Dakota 50 kg BOD_5/ha/day or 600 persons/ha (Allum 1955, Clark and Kalda 1961), Virginia 1200 persons/ha (Cooley 1961), Washington 2500 persons/ha (Berschauer 1961). Other countries like Holland have 60 kg BOD_5/ha/day (Pullen 1973). Melbourne in Australia has 75 kg BOD_5/ha/day (in winter) and 125 kg BOD_5/ha/day (in summer) (Parker, Jones and Greene 1959). Madras City, India has a value less or equal to 100 kg BOD_5/ha/day, or 2500 to 4000 persons/ha (Raman 1972).

However, there is an attempt by many authors to develop international values of organic loading to serve as a guide for sewage pond formulation for all regions. These figures of organic loading are already being used in many countries.

For instance, Caldwell (1946) recommended that organic loading for FP should be less than 55 kg BOD_5/ha/day. Oswald (1961) documented that the maximum surface organic loading (or areal loading) of FP should be 50-60 kg BOD_5/ha/day. Herbert, Neel and Monday (1961) stated that 75 kg BOD_5/ha/day is deemed best for FP. Middlebrooks *et al.* (1979) concluded that the most possible effective value ranges between 25 and 60 kg BOD_5/ha/day.

Hess (1983) recommended that in warm climates the range should be 150 to 400 kg BOD_5/ha/day. Hess also revealed that the lower load is applied to temperature around 20 ^{0}C and the higher one for about 30 ^{0}C. A higher load than this would lead to anaerobic conditions occurring in the ponds and was often observed to cause odorous nuisance. Gloyna (1971) stated that the pond loading should be 150-400 kg BOD_5/ha/day for tropical countries. Metcalf and Eddy (1972) recommended that WSPs, particularly FPs, should have a load of 20-60 kg BOD_5/ha/day.

Nevertheless, there are some operating ponds which still have a higher loading than those recommended. Appropriate examples are those in Bien Hoa, Viet Nam, 750 kg BOD_5/ha/day and Nagpur, India, 500 kg BOD_5/ha/day (Reid 1982). A low organic loading, Herman (1962) pointed out, will result in an unsatisfactory performance. A variation of these surface loadings is viewed as a limiting criterion of nutrient supply to bacteria in the ponds and may affect their capacity to oxidise organic waste. Nevertheless, to a large extent, it is possible that the surface loading of a pond is dependent upon other factors influencing ponds, for example, temperature. For the region with higher temperatures, as in the tropics, higher organic loading is recommended but for a region with a colder climate a lower load is suggested. A current approach to WSP construction is the preference to set up a pilot-plant first to determine a more precise value of organic loading suited to the local environment.

Another design model of WSPs is the MP design equation which is primarily aimed at reducing pathogenic microorganisms. The most frequently used bacterial reduction model is of Marais (Hess 1983) which focuses on decreasing faecal bacteria (*Escherichia coli, Streptococcus faecalis* and others). Marais (1974) characterised bacterial reduction in equation 14 below:

$$N_e = \frac{N_i}{1 + K_b t^*} \qquad \text{Equation 15}$$

where N_i = Numbers of faecal coliform bacteria in influent per 100 ml

N_e = Numbers of faecal coliform bacteria in effluent per 100 ml allowed to discharge into watercourse

K_b = Rate constant for FC removal

t^* = Retention time in days

K_b (at any T ^{0}C) is given by : $k_{b\,(T)} = 2.6\,(1.19)^{T-20}$

where T = ^{0}C

Further if the ponds are arranged in series, then bacterial reduction model will be as follows:

$$\frac{N_e}{N_i} = \frac{1}{(1+K_b t_1^*)\ (1+K_b t_2^*)........(1+K_b t_n^*)} \qquad \text{Equation 16}$$

where: t_1, t_2, t_n = are the detention times (days) in the first, second and n^{th} pond in a series

In general, E.*coli* presence has been used as an indicator of efficiency of wastewater treatment system, and $K_b = 2.0$ per day was used (Gloyna 1971). This value cannot be applied to other organisms e.g. *Salmonella typhi* as documented by Hess (1983).

In summary, most of the design approach models still involve the first order kinetics of organic matter oxidisation which occurs within the ponds. Some authors have included depth and temperature criteria in formulating equations for WSP construction.

There are two interesting points that should be mentioned. First, the temperature effect on pond performance. This later became a part of empirical pond design equations as for McGarry and Pescod (1970), Mara (1976), Yanez (1980), The Asian Institute of Technology (1981) and Arthur (1983). The first authors found linear relationship between air temperature and areal loading after testing in several regions and finally attained an empirical design equation. The latter authors modified the first authors' model by using that model in particular regions of Africa, Asia and South America and eventually derived a new design equation which was expected to have a better performance suitable to that region. Currently, all the above equations are still being used and recommended to any region. There is no definite conclusion on which of those equations is effective over the other.

Second, pond depth and HRT are other interesting factors which some researchers used to create pond design models as in Marais and Shaw (1961) and Mara (1976). These two factors have a major impact on pond function by determining types and rates of biochemical and chemical reactions in the pond. Marais and Shaw (1961) investigated the performance of the pond and found that increasing pond depth can provide a better BOD_5 effluent. This is largely applied to any pond with a heavy load. They also found that whenever HRT increased, the BOD level in the effluent decreased, this HRT factor is similar to Mara's (1976) findings. Design equations with temperature, depth and HRT influences are still being used and some models were found to be operating very effectively, whereas some failed. Some of them are being investigated and applied to a particular location. All of them are very useful for the development of WSPs in various regions.

Regarding the popularity of pond design equations, the temperature oriented formulae are found to have a good application in tropical and sub-tropical zones since these regions have higher air temperature which will assist the rate of biodegradation inside the pond. For the colder climates the models used are more concerned with emphasising pond depth and HRT. An increased HRT in the colder regions is normally found, this is because of sunlight and temperature being limited, so the rate of degradation of organic wastes is much slower than in the tropical and sub-tropical zones.

Also, the surface loading, which is indicated in terms of kg BOD_5 loading per ha per day, (or lbs per acre per day), is still being used by other investigators in forming formulae. Local experience is another way of achieving an organic loading figure for WSP design.

The other alternative is expression in terms of production of liquid waste per capita per unit area of pond. However, in a later approach to WSP construction it was preferred to set up a pilot plant first in order to account for a more precise value of organic loading suited to the local environment. Hydraulic retention time of ponds is also taken into account for creating WSP models.

An attempt to predict the reduction of pathogenic bacteria and other harmful organisms is portrayed in the equation mainly associated with the function of temperature and retention time of the ponds.

3.5 Pond function

According to Tchobanoglous (1991), the WSP system can be classified as facultative, maturation and anaerobic ponds according to the presence of oxygen. The term "facultative" refers to a mixture of aerobic and anaerobic conditions.

In this pond, aerobic conditions are maintained in the upper layers while anaerobic conditions exist towards the bottom. Maturation pond refers to oxygen existing in every level of the pond, whereas the anaerobic pond has no oxygen present.

The purpose of these three ponds is to remove carbonaceous BOD_5, or in other words, sewage organic wastes. Since the sewage plant in this study has no anaerobic pond, the review will present only some significant functions occurring within two pond types: facultative and maturation.

The facultative ponds are to a large extent designed to operate at medium organic loadings whereas the anaerobic pond is aimed at reducing the high content of organics (Tam 1982). It should be noted that each WSP system has been designed according to the characteristics of local sewage, to decide whether it is necessary to include an anaerobic pond or not. Generally, as the liquid wastes enter the FP, settleable solids deposit on the bottom and organic matters here are further anaerobically digested by existing anaerobic-bacteria. The second portion of soluble degradable waste flows into the aerobic and facultative zones. In these two layers, the oxidation reaction of organic wastes is employed by facultative and aerobic bacteria in cooperation with algal photosynthesis which is acting as an oxygen source (Preul and Roesler 1970, Hess 1983).

The role of different species of microorganisms in biological oxidation of organic wastes in the WSP at present, requires further specific investigation. However, there are many fundamental concepts applied to explain these phenomena. There are many schematic diagrams representing biological oxidation-reduction within ponds. Some instances are presented in Oswald (1963), McKinney (1976), Dinges (1982), Mara (1982), Mitchell and Williams (1982), Hess (1983), Tchobanoglous (1991) and it can be concluded from these contributions that there are two fundamental biological oxidation-reduction models occurring in the ponds which are:

(a) $C_aH_bN_cO_dP_e+O_2$ ---> $CO_2+H_2O+NO_3^-+PO_4^-$+New Bacterial Cells, and

(b) $CO_2+H_2O+NO_3^-+PO_4^-$+Light ---> $O_2+C_aH_bN_cO_dP_e$+New Algal Cells.

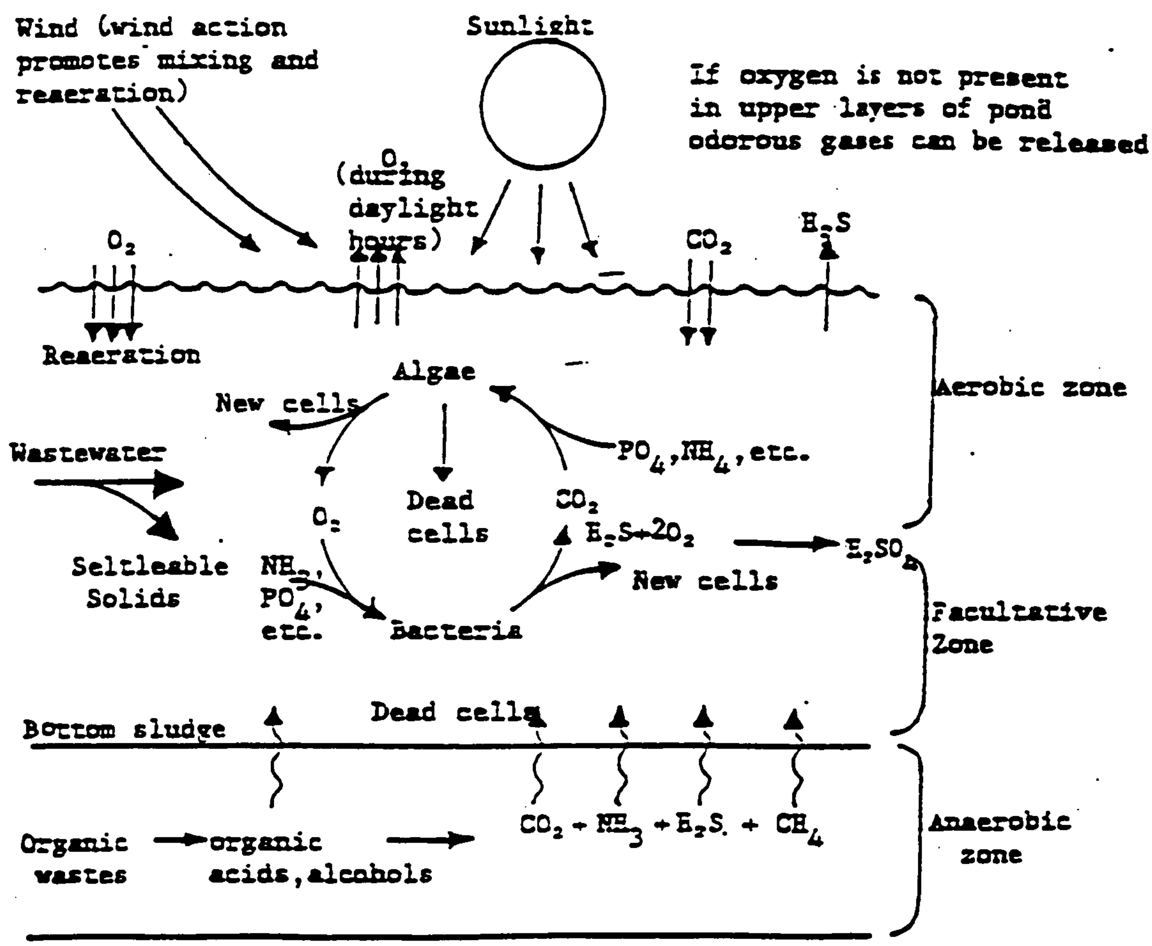
The above equations generally indicate biological reactions which occur in the WSPs, where (a) is the oxidation reaction of organic wastes from sewage by bacteria which use oxygen supply from equation (b), this also gives the end products of carbon dioxide, water, and inorganic material, which are essential to plant growth, including nitrate and phosphate.

The second equation (b) is a reduction reaction where algae utilise carbon dioxide and inorganic substances (mainly nitrate and phosphate - the products from the first equation), to yield oxygen, organic material and new cells of algae. These two reactions present a symbiotic relationship between bacteria and algae and are the most important reactions in reducing organic wastes or BOD_5.

Another phenomenon regarding ponds is the transfer of gases. Oxygen and carbon dioxide gases can be obtained from reaeration and can be released or may penetrate through liquid depth in the ponds (Dinges 1982, Mara 1982, Mitchell and Williams 1982). New cells of bacteria and algae can be part of the suspended solid effluent from the system.

Further, the new organic substances created from reduction processes are then, in turn, used as raw substrates of oxidation reaction whereas the excess amount is discharged as suspended solids (Mitchell and Williams 1982). This model is illustrated in Figure 3.12.

Figure 3.12 Schematic representation of the WSP function.
(Source: Tchobanoglous 1991)

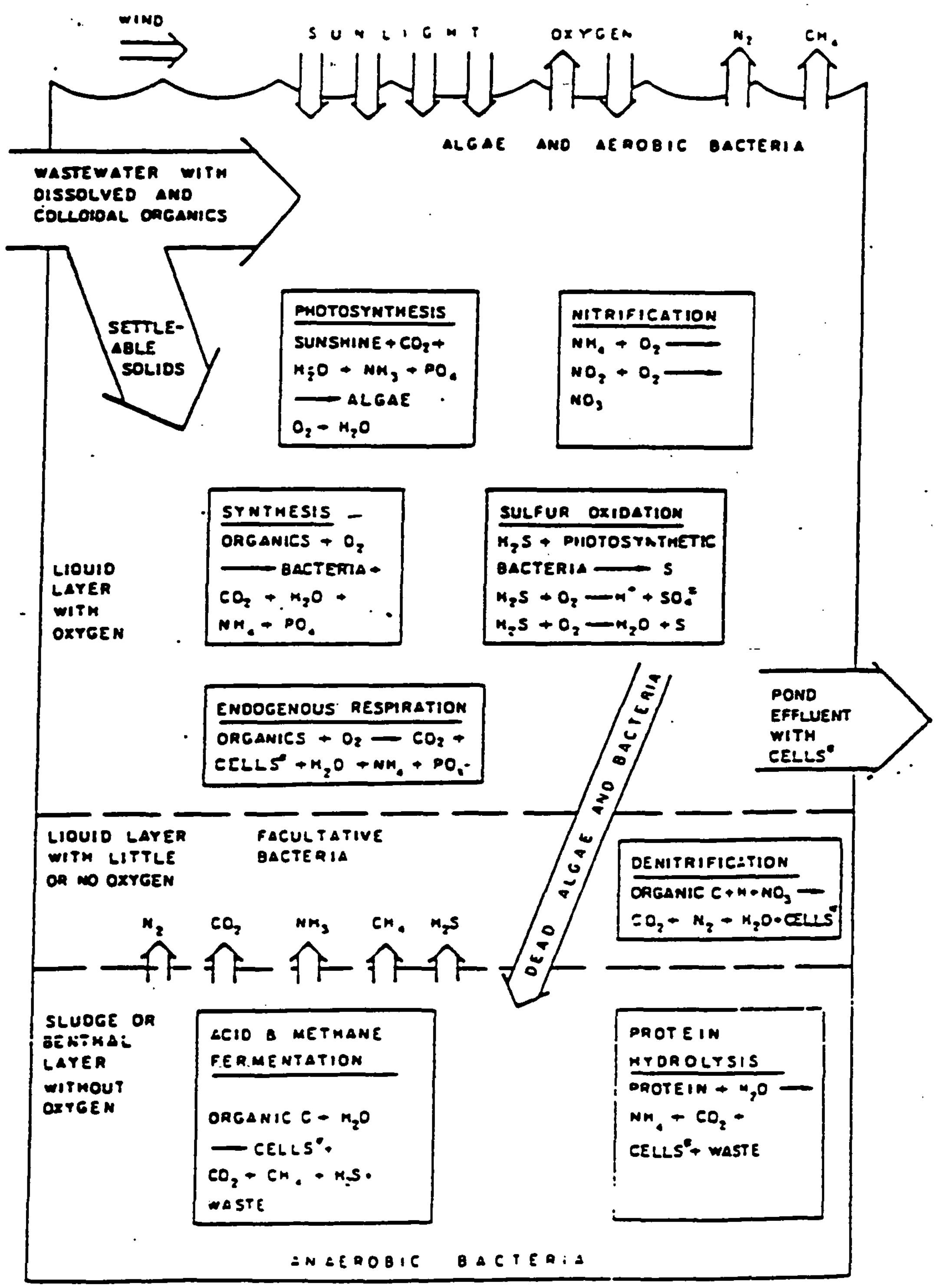


Another interesting model is that of The United States Environmental Protection Agency (1977). This model attempts to explain possible reactions that would occur in the ponds. The pond, as shown in Figure 3.13, is divided into three layers depending upon oxygen conditions. The upper layer is the aerobic zone, the intermediate is facultative and the last one is the anaerobic zone. This model tries to show several reactions happening from the upper to the bottom layers. These reactions still mainly involve bacterial and algae oxidation-reduction reactions in the ponds.

However, the rate and direction at which these reactions actually happen in the ponds have not been widely documented. Most of these equations still require further examination. Largely, such biological degradation reactions in FP are quite varied and dependent on external environments, e.g. sunlight, wind, depths of ponds, temperature or even characteristics of sewage. Nevertheless, the common agreement among many authors is still centred on a symbiotic relationship in oxidation-reduction between bacteria and algae which may be viewed as the most critical point for the system.

In summary, the WSPs have a function similar to a biological reactor in a conventional wastewater treatment process. However, WSP is largely a unit process employing wholly natural processes for the treatment of domestic organic wastes which enter the unit. These domestic organic wastes are further biologically oxidised by bacteria from organic into inorganic forms. This mineralisation process will further benefit algae as a nutrient in the reduction process. Such basic reactions are viewed to have an important role in reducing organic wastes. For MPs, the expected reactions are similar to those in the aerobic zone of FPs. Since the ponds' functions are mainly related to external environment, these functions can be further expressed in association with factors affecting pond performance. The following topic discusses factors affecting pond performance.

Figure 3.13 The biochemical reaction occurring in WSPs.
(Source: US. Environmental Protection Agency 1977)



NOTE - *REPRESENTS ALGAL AND BACTERIA CELLS

3.6 Factors affecting pond performance

For WSP systems to achieve high performance, there are many factors involved with the ponds' processes. These factors are organic loading, detention time, temperature, light, liquid mixing, wind action, pH and dissolved oxygen.

As mentioned in the former topic, recommended designs for WSPs using organic loading are essential to achieve a high quality of effluent from WSP systems. This is considered on the basis that every WSP has limitations in treating liquid wastes. For example, Stouse (1964), and Canter, Englande and Mauldin (1969) found that coliform counts from effluent increased proportionally with higher organic loading. Regan and McKinney (1977) stated that the effective operation of a WSP is dependent on a limited organic loading. They found BOD_5 removal efficiency of WSPs reduced from 90% to 80% when organic loading increased from 60 to 250 kg BOD_5/ha/day. Hess (1983) stated that if organic loading of WSPs was over 400 kg BOD_5/ha/day, an often unwanted odour can be detected and the efficiency of WSPs to remove organic wastes is reduced. Such decrease in efficiency of WSPs, resulting from overloaded organic loading, is also documented by many authors like Barsom (1973), O' Brien (1978), Mara (1982) and Banerji and Ruess (1987).

Hydraulic retention time (HRT) is another important factor influencing pond performance. HRT is the theoretical time that sewage will be retained in the ponds. This is derived from pond volume divided by average flow. For a better WSP performance, Caldwell (1946) found that the appropriate HRT value is at least 25 days. Allum (1955) noted that HRT of WSP should be 30 days. Pipes (1961) noted that WSPs should have HRT not exceeding ten days. Metcalf and Eddy (1972) stated that for FP, HRT should be 7 to 20 days. Ansari (1973) investigated an optimum HRT result of 20 to 30 days. For the tropical zone, Reid (1982) documented that HRT for the pond should range between 17 and 33 days. There are still doubts as to whether there is a specific detention time effective for certain areas or not. Even so, there are many suggested HRTs for WSP systems.

General approaches regarding the effect of HRT on a WSP's performance may be evidenced by the works of many investigators. For example, Meron, Rebhun and Sless (1965) showed a very interesting result.

By increasing HRT, there were significant decreases in BOD_5, suspended solids (SS), nitrogen, phosphorous and coliform bacteria. Franzmathes (1970) revealed that the major factor influencing bacterial die-off rate is HRT. Bucksteeg (1987) pointed out that the coliform count decreased by two orders of magnitude when the HRT was nine days, but that one day of HRT does not lead to considerable coliform removal.

Fall (1971) published that BOD and SS reduction were functions of HRT, so increasing HRT will maximise BOD and SS reductions. SS were found to be in greater number whenever HRT was increased (Reynolds 1975). Chaudhuri (1973) found that a longer HRT provided more efficiency in reducing virus concentration in the effluent. Oswald *et al.* (1964), as well as Bokil and Agrawal (1977) examined the relationship between algal culture and HRT in the WSPs and noted that algal concentration increased proportionally with decreasing HRT. Mayo (1989) found that bacterial mortality rates were more rapid with an increase in HRT and a shallower pond.

Dinges (1982) documented that there is usually a wide disparity between the theoretical and the real HRT. This was a consequence of problems like short-circuiting, seepage losses, evaporation and reduction of pond capacity due to sludge accumulation. Further, in practice, like the WSPs in the USA, there are varieties of HRT ranging from 20-180 days from 50 states (Canter and Englande 1970).

Different HRT practices mentioned by many authors depend on organic loading and other physical and chemical factors. For a region with a high load, a higher HRT is recommended since the rate of biodegradation of organics by microorganisms is a function of time, following the biooxidation kinetic reaction (equation 1). To achieve a better effluent quality, providing more days of HRT is essential. This is in contrast to a place with lower loading which needs less HRT.

Environmental factors, such as temperature, are further limiting factors which will determine the rate of biochemical and chemical reactions inside the pond. With the same high load, for the place located in the tropical zone the HRT needed will be less than for the colder zone. This relates to the degree of degradation of organic wastes as its rate is temperature dependent. Many documents show that when HRT is constant, in summer with higher temperatures, the rate of biooxidation increases, whereas in winter this rate drops.

To discuss the reduction of SS in the pond in summer by increasing HRT is quite ironical. The fact is that whenever the rate of biodegradation increases the new bacterial and algal cells, which are the products of that reaction, will also maximise. Eventually these two residues are in turn contributing another form of new waste input to the pond and result in a high level of SS. So, SS reduction inside the pond can be occurring when such residues settle to the bottom providing the upper water column has a low SS concentration. This often occurred when the temperature dropped during the day. Such settled solids become the organics for degradation by anaerobic bacteria at the bottom zone.

Another possibility of SS reduction is that during summer with high temperature the algal mats appear to be floating up to the surface layer of the pond and the operators normally remove those mats from the pond. This also causes the level of SS to decrease. As high temperatures lead to an increase in the biooxidation rate and with the algal mats not yet being removed from the pond, the SS level in the pond will fluctuate as it depends on the degree of SS settling to the bottom.

With high temperature the rate of SS settling is low. Also, the gases produced from the anaerobic reaction at the bottom zone will influence the dispersion of SS back to the upper layer of the pond. In contrast, when the temperature drops the rate of SS settling is high, this leads to low levels of SS found in the pond water column. So, the reduction of SS level in the effluent depends on the main factors above.

For the above reasons, given the scenario of tropical climate regions, increasing HRT does not necessarily lead to SS reduction. This point on HRT affecting SS reduction can be found to be a matter of disagreement mentioned in Fall (1971) and Reynolds (1975).

Another important point relating to HRT is that bacterial population die-off rate is found at a higher rate whenever HRT is increased as Mayo (1989) mentioned. This matter of bacterial die-off rate associated with HRT also needs to be taken into account in conjunction with temperature. In fact, the bacterial die-off rate is high when HRT is increased, this can be explained mainly by bacterial nutrients and temperature conditions. In general, the bacteria can grow in any environment if (1) there are sufficient nutrients available, (2) there is an absence of toxic substances and (3) the environment itself is suitable.

Given the fact that at any WSP plant with a multi-cell arrangement the subsequent pond thus has a higher HRT. For example, as in the last pond, the nutrient for bacterial growth is limited since organic matter is almost degraded in the preceding pond. Thus, the bacterial growth rate is low. To increase HRT can then result to bacterial population decrease as from the nutrient matter mentioned. Increased temperature can affect the bacterial die-off rate resulting from solar radiation. Many authors for example Mayo (1989) found that with high temperatures, the die-off rate of bacteria is also high.

Temperature affects the reaction rate occurring inside the pond. Metcalf & Eddy (1972) documented that temperature is essential in determining the degree of biodegradation of organics by microorganisms. This rate increases when the temperature is raised and will double with every 10 ^{0}C.

However, this is not always the case, if taking into account limited nutrients so the rate of biodegradation by bacteria will not necessarily follow the temperature increase. Further if the sewage contains toxins such as cyanide, raising temperature will increase the solubility rate and consequently affect the bacterial die-off. Also, the degradation rate happens over a specific range of temperature. Whenever the temperature increases beyond a certain level, the bacterial will also die.

Middlebrooks *et al.* (1979) noted that pond temperature at or near the surface will, to a large measure, determine the success of predominant species of algae, bacteria and other aquatic organisms. Looking at the early development stage of WSPs, Pipes (1961) recommended that for a better performance of WSPs, the temperature should be maintained as high as possible and should not be allowed to fluctuate excessively. Anderson and Zweig (1962) also found an effect of temperature on algal species. The predomination of algae in the ponds was dependent on temperatures within the ponds. For instance, 30-35 ^{0}C for the green and 35-40 ^{0}C for the blue-green algae. Tariq and Ahmed (1981) examined algal flora in WSPs and found that if the pond temperature was increased beyond 30 ^{0}C, green algae was replaced by blue-green algae.

Middlebrooks *et al.* (1979) quoting from Macais (1962) reported that *Chlamydomonas* spp. was growing profusely at a pond temperature range of 29-34 ^{0}C. Reid (1982) cited from Diaz (1975) that the optimum temperature for the pond system should be ranging from 25-32 ^{0}C.

Several authors have agreed (Parker 1962, Oswald 1972, McKinney 1976, Arthur 1981, Hess 1983) that whenever temperature increases, WSPs show an improvement in performance. The prime source of increased pond temperature, in this case, is that of solar radiation. Post (1970) found that temperature had a more crucial effect on *E.coli* than on the other coliform groups, since *E.coli* is more sensitive to high temperature with a greater die-off rate.

Lakshminarayana and Abdulappa (1973) also documented that helminth populations in WSPs were affected by pond temperature. But Marais (1970) concluded that solar radiation did not seem to be a critical factor for algal growth and oxygen production in the ponds of tropical areas. Consequently, Marais and Shaw (1961) had earlier designed a pond formula (equations 4 and 5) which emphasised depth and retention time. Others, like McGarry and Pescod (1970), who concentrated on the effect of temperature, in contrast created another model (equation 7) directly related to temperature.

According to Hess (1983), the efficiency of facultative and maturation ponds will be better during a sunny day with a blue sky, and when air temperature exceeds 20 ^{0}C with a mild wind. This will obviously enhance photosynthetic activities of algae in producing oxygen supply to ponds. However, in very high temperatures of summer, Oswald *et al.* (1964) noticed that there was likely to be an occurrence of algal bloom on the surface of the pond and this would cause the lower part to darken, consequently reducing photosynthetic oxygenation inside the ponds.

Hess (1983) documented that dissolved oxygen seemed to be released to the atmosphere when the temperature was raised, particularly if supersaturation occurred. He also noted that ammonia-nitrogen and phosphorus were likely to increase in the colder months, and in high temperature circumstances, blue-green algae (*Cynanophyceae*) seemed to grow profusely, replacing green algae. This also led to aerobic bacteria consuming a plentiful amount of oxygen, and resulted in anaerobic conditions in some locations within the pond.

Towne, Bartsch and Davis (1957) stated that solar radiation was an essential factor in algal photosynthesis, thus supplying oxygen to the ponds and further activating microbial oxidation in the ponds. However, Hess (1983) stated that the rate of photosynthesis is affected by solar intensity only to a certain point. If excessive light intensity is beyond that level, the photosynthesis rate will remain constant.

For harmful bacterial reduction, Mayo (1989) emphasised that the bacterial die-off rate in the WSPs is directly correlated to solar radiation. Ohkaki, Ketratanakul and Prasertsom (1986) also documented that sunlight can deactivate enteric viruses especially when they are near the top layer of the ponds. Moeller and Calkins (1980) found that ultraviolet rays from sunlight had a critical effect on coliform mortality. Tariq and Ahmed (1981) investigated light intensity and BOD reduction in WSPs and found that WSPs have an optimal capacity in reducing BOD with a range of light intensity ranging from 49,000 to 63,000 lux, but as light intensity increased further to 77,000 lux, the efficiency of WSPs in BOD removal was decreased.

The pH is also important to the life of microorganisms. Most of the microbes cannot tolerate pH levels above 9.5 and below 4. The optimum level for microbial growth ranges between 6.5 to 7.5 (Metcalf and Eddy 1972). Pipes (1961) suggested that pH of WSPs should be 7 - 8. Pipes (1962) found that when there was a lowering of pH in the influent, BOD_5 removal increased, particularly in the pond with a short retention period. However, this was not found to be significant for ponds with a retention period of more than 20 days. Sebastian and Nair (1984) found that there was an increased *E.coli* mortality when pH rose beyond 9.4.

Middlebrooks *et al.* (1979) summarised a relationship between pH and photosynthesis activities inside ponds. Soluble carbon dioxide is consumed by algal photosynthesis and this later leads to an increased pH value (the hydrogen ion concentration). This can be illustrated by equation 17 below:

$$CO_2 + H_2O \longleftrightarrow H_2CO_3 \longleftrightarrow HCO_3^- + H^+ \qquad \text{Equation 17}$$

On the other hand, at the bottom layer which maintains an anaerobic condition, pH is found to be lower because of volatile acids produced by acid-forming bacteria. This acid will be neutralised by bicarbonate alkalinity (HCO_3^-) in the ponds. Hydrogen sulfide and other odorous gases were detected if pH was low (Pipes 1961, Oswald *et al.* 1964). Parker, Jones and Greene (1959) and Parker (1970) investigated the largest WSPs in Melbourne and other sewage lagoons in Australia. It was found that odorous problems arose whenever pH was not controlled. This was happening especially in anaerobic ponds.

Hess (1983) also observed that the colour of FPs can imply something about the pH in the ponds. Whenever an FP was deep-green in colour, it can be estimated that pH in the ponds is in the alkaline range. On the other hand, if the FP had a yellowish green or milky appearance, acidity in the pond increases. Meenaghan and Alley (1963) and Hess (1983) also found that pH in the ponds varied during the day. For instance, pH of the pond is quite low in the morning as a consequence of an excess of CO_2 produced by aerobic bacteria during the previous night, and pH will increase in the afternoon due to photosynthetic activity by CO_2 consuming algae.

Dissolved oxygen (DO) is another prime factor affecting WSP performance. Bartsch and Allum (1957) noted that the rate at which organic matter in the pond is being decomposed is dependent upon the amount of prevailing oxygen. Also, Malina (1973) found that BOD, COD, enteric bacteria and viruses are more effectively treated by WSPs if DO level is increased. Caldwell (1946) found that DO levels in WSPs are lower at night, early morning and rise up to a high peak in the afternoon.

Further, Herman and Gloyna (1958), also in Meenaghan and Alley (1963) both stated that DO levels varied almost directly with incidental light. In addition, Mara (1982) documented that the oxygen condition will determine types of bacteria in the oxidisation of organic matters. For instance, at the bottom, acid and methane-forming bacteria are the major groups while facultative bacteria are in the facultative zone and at the top layer of the pond, are the aerobic bacteria.

Bokil and Agrawal (1977) reported that the amount of DO in WSPs decreases inversely proportional to pond depth. Abeliovich (1982) investigated that DO at the upper layer of WSP water columns is essential for trapping sulfide which oxidises from the anaerobic processes at the bottom. Thus, it can prevent any nuisance emerging from the WSP plant.

There are two main sources supplying oxygen to the ponds: surface reaeration and photosynthesis. Photosynthesis is no doubt the main source of oxygen supply to the ponds. For surface reaeration, Pipes (1961) and Oswald (1963) noted that this type of oxygen generation is of minor importance to pond oxygenation compared to photosynthesis. However, Ellis (1983) reasoned that such surface reaeration is crucial for WSPs. This is also substantiated by Mara (1982), who observed that often at night DO level in the pond is nearly zero so that wind-induced reaeration is necessary in order to prevent a noxious condition.

The former investigators like Pipes (1961) recommended that the location of a WSP should not be along wind direction. This is because he viewed it necessary to keep the temperature warm inside the pond to maximise biological oxidation without heat loss from wind induction. In contrast, Mara (1976, 1982) and Ellis (1983) have shown that wind-induced reaeration has an important role in supplying oxygen to ponds. It can reduce thermal stratification in the water column, help liquid mixing, supply oxygen at night time, or even alleviate the movement of algal culture in ponds. However, another experience noted by Hess (1983) is that a windy location is often found to cause strong waves which bring about embankment erosion of the ponds.

Depth is another limiting factor in pond performance. Chaudhuri (1973) revealed that a shallow pond has better capability of reducing viruses than a deeper one. Sarikaya, Saatici and Adulfattah (1987) documented that the pond depth has a significant effect on the coliform die-off rate. Sarikaya and Saatici (1988), and Mayo (1989) found that bacterial die-off rate is inversely proportional to pond depth.

Seasonal variation is found to be a constraining factor affecting pond operation. Mackenthun and McNabb (1961) examined BOD_5, N, H_2S and total suspended solids (TSS) removal in summer by WSPs and found that the level of those contaminants was higher in hot seasons than in winter.

Smallhorst (1961) insisted that pond performance is best in summer. Stouse (1964) noted that in winter he found BOD_5 above 20 mg/L, NH_3 12 to 20 mg/L, but in summer the amount of BOD_5 dropped less than 20 mg/L and NH_3 is 0 to 7 mg/L. Willford and Middlebrooks (1967) stated that total dissolved solids (TDS) level in the WSP varies with seasonal variations. Fisher (1968) also documented that pond performance was significantly dependent on climatic conditions. McGriff (1981) concluded that BOD and SS varied in sensitivity to climatic and seasonal conditions. Santos and Oliveira (1987) noted that nitrogen transformation, for instance, nitrite to nitrate, is greater in summer. Henry and Prasad (1985) examined BOD_5 reduction by WSP plants in Canada and found that BOD_5 percentage removal in summer was 90% and 80% in winter. Heavy metals (Cu, Ni, Zn, Cd, Pb and Hg) were found to have a greater degree of reduction in summer than in winter (Saniassy and Lemon 1986). Bucksteeg (1987) cited from Schleypen (1985) found that PO_4 reduction was higher in summer at 63% whereas in winter it was 19%.

3.7 Physical, biochemical and bacterial tests required for WSP performance evaluation

To evaluate WSPs performance in compliance with environmental quality control, especially aimed at protecting receiving waterways from pollution, Middlebrooks *et al.* (1979) recommended that tests of 5-day biochemical oxygen demand (BOD_5), total suspended solid (TSS) and acid-base (pH) are the primary parameters required to be analysed. Widmer (1981) suggested methods of WSP assessment to WHO member countries, similar to that of Middlebrooks *et al.* (1979), examining of BOD_5, TSS, chemical oxygen demand (COD), total organic carbon (TOC), pH, dissolved oxygen (DO), and other determinants required by each country's effluent standard such as phosphate (PO_4), ammonia-nitrogen (NH_3-N) and nitrate-nitrogen (NO_3-N). However, for TOC, Widmer (1981) stated that this parameter is optional since this test requires sophisticated procedure and equipment.

Hess (1983) noted, like Widmer (1981), that TOC is optional since BOD_5, COD and TOC are aimed to measure carbonaceous organic content in liquid wastes inflow and outflow from WSPs. Therefore, these three parameters can be used interchangeably. McKinney (1976) documented that BOD_5 is an initial parameter for evaluating WSP efficacy. Hence, BOD_5 is used to measure biodegradable organic matter while COD represented the total oxidisable organic component. Then, McKinney (1976) viewed that these two determinants were necessary for conducting WSP performance tests. McKinney (1976) also documented that BOD_5 in general, is 58 percent of COD with 5 percent variation.

However, this ratio is not an absolute rule since such a ratio is dependent on the characteristic of the sewage and source of sewage generated. Industrial wastes have a certain composition of sewage which can contribute a certain ratio of BOD_5:COD. For domestic sources, a variation both in quality and quantity of sewage is often found, so the ratio does not necessarily follow McKinney's.

Cosser (1982) noted that such an evaluation of WSP performance, applying the method of COD test, is more credible measure than BOD_5 since most organic matter content in the pond is algae. Then Cosser (1982) stated that a BOD_5 test does not provide a correct figure of WSP performance. Such a comment from Cosser (1982), nevertheless, is still in contention among investigators. The above statement is viewed as the only limitation of the BOD_5 test but does not imply that the COD test is more useful than BOD_5 test.

Expressions of WSP performance evaluation also include tests carried out in several current studies. Most of these current studies of WSP performance are directed at investigating its efficiency in (1) complying with effluent standards and (2) controlling pollution in the receiving waterway.

The works commonly centre on BOD_5, COD, pH, NH_3-N, PO_4-P, TSS, temperature, DO and faecal coliform count tests. Most of the authors view that these parameters are significant in determining the qualities of the receiving water body. This can be shown by the latest studies in WSPs performance, for example in France, Boutin, Vachon and Racault (1987) conducted tests specifying BOD_5, COD and TSS. In another, Bucksteeg (1987) evaluated WSP efficiency in Germany using BOD_5, COD, NH_3-N, PO_4-P and faecal coliform counts.

Banerji and Ruess (1987) made an evaluation of WSP efficiency in Missouri and Kansas Cities in the USA by applying BOD_5, TSS, pH and temperature measurements. In Canada, Mathavan and Viraraghavan (1991) focused tests on BOD_5, TSS, NO_3-N, PO_4-P and total coliform. Yhdego (1992) in Tanzania, emphasised testing on BOD_5, NH_3-N, PO_4-P, TSS, temperature, DO, and faecal coliform counts.

3.8 Case histories of pond performance

The major approach for investigating pond performance is presented according to determinants of removal efficiency. Such pollutants concerned are BOD_5, TDS, TSS, N, P, bacteria and viruses. These are, according to Widmer (1981), viewed as the main group of parameters affecting the received waterway pollution and which can be used for evaluating WSP performance. However, BOD_5 (mg/L), N (mg/L), P (mg/L), TSS (mg/L) and *E.coli* (MPN/100 ml) are common essential parameters which require investigation and have been used in research as illustrated in Table 3.3 and Table 3.4.

Reported results of WSP performance are mostly expressed as percentages in pollutant removal efficiency. Most of the WSP performance reports are mainly from Western countries including Australia. There are a few publications from developing countries such as India, Tanzania and Malaysia. Case histories here are in fact intended to provide a picture of the general performance of WSPs rather than to provide a deep investigation.

Table 3.3 shows some WSP performances in various regions which are mainly investigations of the removal efficiency of major contaminants (physical determinants-TSS and TDS, inorganic determinants-N and P and an organic determinant-BOD_5).

Table 3.3 Case studies of WSP performance with respect to percentage reduction of BOD_5, TSS, TDS, N and P.

Determinants	Percent removal	Location of WSPs	References
BOD_5 (mg/L)	70-99	USA (Oklahoma)	Assenzo and Reid (1966)
	70-90	USA	Bartsch and Allum (1957)
	90-95	USA (Washington)	Berschauer (1961)
	70	India	Bhaskaran and Chakrabarty (1966)
	90	USA (Louisiana)	Canter, Englande and Mauldin (1969)
	80	USA (Virginia)	Cooley and Jennings (1961)
	90	USA	Gloyna and Aquirre (1970)
	87	USA (Pennsylvania)	Sido, Hartman and Fugazzotto (1961)
	18	Jordan	Salem and Lumbers (1987)
	80-90	Canada	Henry and Prasad (1986)
	79	Malaysia	Bradley (1983)
	94	USA (Missouri)	Grewis and Burkett (1966)
	70-90	USA	Herman and Gloyna (1958)
	74-86	USA (Ohio)	Horning (1964)
	77	USA	Klock (1972)
	85	USA (Kansas)	Lyman (1970)
	92-98	USA (Texas)	Martin *et al.* (1976)
	90	USA	Novak (1976)
	99.1	USA	Chaffin (1976)
	98.2	USA	Orgeron (1976)
	83	USA (California)	Oswald (1976)
	93	Mexico	De la O and Martinez (1976)
	60-92	USA (Missouri)	Bernaji and Ruess (1987)
	75.1	Tanzania	Yhdego (1992)

TSS (mg/L)	81.1-90.8	USA	Fall (1971)
	50	Malaysia	Bradley (1983)
	54	USA	Novak (1976)
	24.3	USA	Chaffin (1976)
	-44.3	USA	Orgeron (1976)
	42	USA (California)	Oswald (1976)
	87.5	Mexico	De la O and Martinez (1976)
	50-78	USA (Missouri)	Banerji and Ruess (1987)
	71.4	Tanzania	Yhdego (1992)
TDS (mg/L)	21-39	USA	Bush, Isherwood and Rodgi (1961)
N (mg/L)	30	USA (Oklahoma)	Assenzo and Reid (1966)
	39-43	USA (Missouri)	Neel, McDermott and Monday (1961)
	75	India	Rao and Agrawal (1973)
	31	USA	Novak (1976)
	58	USA (California)	Oswald (1976)
	89.2	Tanzania	Yhdego (1992)
P (mg/L)	95	USA (Oklahoma)	Assenzo and Reid (1966)
	90	USA (Kansas)	Lyman (1970)
	94-98	USA	Neel, McDermott and Monday (1961)
	47	USA	Novak (1976)
	26	USA (California)	Oswald (1976)
	75	Germany	Bucksteeg (1987)

The above data in Table 3.3 indicate WSP performance emphasising major pollutant reduction. In general, WSPs can perform well in organic waste reduction such as BOD_5, P and N. In the 1970s, there was growing concern about stringent environmental standards and in 1974 the USA established secondary environmental standards. Some of the WSP effluents in the USA did not meet standards, particularly for TSS (not exceeding 30 mg/L). For instance, three WSPs in South Florida were evaluated by The Department of Environmental Regulation who found that TSS levels in those plants were outside the secondary effluent standards (Urteaga 1990).

A three-cell FP in Mississippi was assessed and found to contain TSS and coliform counts above the standard (Hill and Shindala 1990). TSS also in Eudora, Kansas City, USA, did not meet the standard due to the high content of living and dead algae (McKinney 1990). However, another group of WSPs achieved better performances which met standards, e.g. in South Texas (Urteaga 1990) and 49 lagoons in Michigan, USA (Pierce 1974). An initial conclusion made by McGriff (1981) and Urteaga (1990) is that while some WSPs do not meet the standard with TSS levels, some do. This might be due to geographical differences or climatic and seasonal conditions which influence the performance of the WSPs.

Another important consideration in WSP performance is bacterial reduction. Since this is a prime hazard to public health, WSPs are expected to reduce pathogenic bacteria. The bacteria index normally used is coliform bacteria. Table 3.4 shows the effectiveness of some WSPs in decreasing coliform bacteria.

Table 3.4 Effectiveness of some WSPs in removing coliform bacteria.

Percentages of Bacterial Reduction	Locations	References
88-99.9	Oklahoma, USA	Assenzo and Reid (1966)
93-99	Northern Plain, USA	Bartsch and Allum (1957)
99.9	Washington, USA	Berschauer (1961)
98.1	Louisiana, USA	Canter, Englande and Mauldin (1969)
90-99	Virginia, USA	Cooley and Jennings (1961)
85.9-97	Ohio, USA	Horning (1964)
99	USA	Kabler (1959)
90	USA	Oswald *et al.* (1964)
99	Pennsylvania, USA	Sido, Hartman and Fugazzotto (1961)
70-95	Dakota, USA	Towne, Bartsch and Davis (1957)
99.99	Developing Countries: WSPs sponsored by World Bank.	Bartone and Arlosoroff (1987)
95	Kuwait	Esen (1987)

Considering the rate of coliform die-off in WSPs, there is agreement among authors like Meron, Rebhun and Sless (1965) and Klock (1971). They found that coliform die-off mechanisms in WSPs followed Chick's Law. Chick's Law is illustrated as follows (Barnes and Wilson 1978):

$$\frac{N_t}{N_o} = \exp(-kt)$$

where N_t and N_o refers to bacterial numbers per unit volume at times t and $t = 0$ respectively, and k is a proportionality constant with units of t^{-1}. For k value of *Escherichia coli,* Eckenfelder (1970) noted that its value is 0.24 and other species have their own values. WSPs have been viewed so far to have a high potential for reducing coliform bacteria over other wastewater treatment systems (Anderson and Zweig 1962, Hess 1983, Mara 1988) shown in Table 2.5. Not only the coliform group, but also the enteric bacteria, which are resistant to antibiotic drugs, are found to be successfully treated by WSPs (Walter and Vennes 1985).

Viruses and tuberculosis bacilli have also been shown to be effectively removed by WSPs (Sobsey and Cooper 1971, Viraraghaven 1973, Ohgaki, Krtratanakul and Prasertsom 1986). Parasitic eggs and cysts, which can cause human intestine infections in developing countries, were not found in WSP effluent (Lakshminarayana and Abdulappa 1973, Lansdell 1987). It is interesting to note that such eggs of parasites were later found to be accumulating in sludge at the bottom of WSPs (Schwartzbrod, Bouhoum and Baleux 1987). Sludge management is now becoming an important topic in WSP studies (Farrell 1984).

3.9 Trouble shooting, maintenance and operation of WSPs

The problems mainly associated with WSPs are those involving odours, vegetation, mosquito breeding, bank erosion, and failure to meet effluent standards.

Experiences from early WSP plants showed that odour problems commonly occurred in this type of plant. For instance in the USA, WSP plants in South Dakota (Clark and Kalda 1961), Nabraska (Filipi 1961), Mississippi (Johnston 1961), West Virginia (Weigand 1983), or in Norway, (Odegaard, Balmer and Hanaeus 1987). Johnson (1961) and Porges (1963) noted that odour nuisance often generated from WSPs mainly due to dead blue-green algae. Neel, McDermott and Monday (1961), Shilling (1963), and Odegaard, Balmer and Hanaeus (1987) further added that such a problem is also generated whenever there is excessive organic loading in the ponds. Bucksteeg (1987) experienced the same odour nuisance from most German FPs and pointed out that it resulted from accumulated sludge in the ponds. Lumbers and Andoh (1987) noted that odour problems of WSPs were also related to seasons, particularly hot seasons.

Regarding odour nuisances, Odegaard, Balmer and Hanaeus (1987) suggested that aeration of WSPs would be a way of reducing the problem. Multi-cell WSPs are recommended by Bartone and Arlosoroff (1987) as capable of dealing with odours.

George (1982) documented that using intermittent sand filters could upgrade WSP performance and could reduce odour from WSP effluent. Bucksteeg (1987) found that pigpen odour could be reduced by removing sludge from the ponds.

Weeds growing alongside the pond embankments are another problem in the operation of WSPs. These weeds are also associated with mosquitoes since they act as a breeding place. WSP plant operation has frequently encountered this problem whenever there is poor control of vegetation (Beadle and Harmston 1958, Clark and Kalda 1961, Rapp and Emil 1965, Gerardi and Grimm 1987, Pescod and Mara 1988).

Design factors also cause enrichment of weeds e.g. Rapp and Emil (1965) found that too flat a side slope and irregular shapes of embankment caused the harbouring of weeds and insects. Okoronkwo and Odeyemi (1985) also noted that poorly designed WSPs in Tanzania were major breeding places for insects and could become a serious health hazard to the public. Kitterrle and Enns (1968) documented that there were at least 60 species of insect in WSPs, in Missouri, USA. Insect transmitted diseases in developing countries which are considered important are for example, diarrhoeal diseases and blood infections etc (Dr Pragom Pangvudthipong, Department of Health, Thailand, Pers. Comm.). Routine maintenance by mowing vegetation is recommended to be the most effective means of control (Pescod and Mara 1988).

However, some aquatic weeds were found to be aiding the reduction of P and N since Harvey and Fox (1973) examined duckweed (*Lemna minor*). It can decrease P and N, 86% and 67% respectively from the liquid waste in WSPs. Though this kind of weed can aid the removal of nutrients from sewage, there is still a need for further studies regarding the appropriate population of duckweed in the pond suitable for efficient removal. Boutin, Vachon and Racault (1987) found duckweed in France proliferating and covering pond areas causing undesirable results. Radziej (1988) also found that too many aquatic macrophytes could lead to less efficient WSPs.

Water Hyacinth (*Eichhornia crassipes*) growing in WSPs is another aquatic plant which can aid BOD and SS reduction (Stewart 1979, Wolverton 1979, Santos 1987, Orth and Sapkota 1988). However, DeBusk, Reddy and Clough (1989) stated that *Eichhornia crassipes* can also raise DO levels in the ponds.

The important point here to be noted is that even aquatic plants can uptake P and N but whenever they die P and N can release back to the pond water column, so the removal of such plants from the pond is needed to reduce the soluble P and N contents of the pond.

The correct number of water hyacinths and duration of harvest are still being investigated by many researchers. Looking at the application of aquatic plants to pond systems may upgrade WSP efficiency in the purification of sewage, and also be useful for irrigation or reuse of effluent for agriculture.

Bank erosion is another concern in a WSP's operation. This trouble is frequently caused by waves generated from wind induction (Hess 1983). Many authors, such as Rapp (1960), Filipi (1961), Villiers and Farrell (1977) and Weigand (1983) reported having WSP embankment failure by waves, and pointed out that such circumstances also resulted from poor design. Mara (1976) documented that in solving this problem, there should be careful designing of embankment slopes.

Hess (1983) insisted that location of ponds towards direction of winds should be taken into account. Rapp (1960) suggested that planting grasses covering the bank would be necessary and should be routinely maintained. Villiers and Farrell (1977) invented fibre mats covering embankment slopes and found that such mats could prevent erosion. Ellis (1981), Pescod and Mara (1988) noted that to avoid erosion of slopes, concrete slabs should be added thus protecting the embankment from erosion.

One problem with WSP performance is the inability to provide a TSS value which meets the standard. The development of WSPs especially until the early 1970s dealt with BOD, N and P incredibly well and had far greater efficiency than other systems (Gakstatter 1978). However, since the emergence of a new concern for high quality effluent and the establishment of new standards in the mid-1970s in many countries e.g. USA, Australia etc., WSPs have suffered from a high content of TSS in the effluent. This is because TSS in WSPs is mostly composed of algae. Several researchers are, then, trying to solve this problem. For instance, in Australia, the Bureau of Environmental Studies, Canberra, ACT, sponsored a project in 1975 on pond-harvesting technology. To achieve algae separation from sewage ponds, a model machine has been developed to remove both colloidal and settled algae (Department of Environment, Housing and Community Development 1975). Bernhard and Kirchgessner (1987) have surveyed 612 WSPs in France which were built in 1983-1986 and found that clogging in WSPs is a consequence of high TSS content.

Other studies for improving TSS (algae removals) in the WSP effluent are: by using alum sulfate (McGarry 1970, Parker 1973), upflow filtration (McGhee and Patterson 1974), biogrowth sheets (Wixson 1975), infiltration (Dornbush 1976), rapid sand filtration (Boatright and Lawrence 1977, Sanks 1977) and intermittent sand filtration (Harris 1977).

Many reports for upgrading pond effluent suggest using aquaculture such as applying a microcrustacean polyculture system (Kawai 1987), using polyaquaculture (McGarry 1982), using native fish, like fathead minnow (*Pimphales promelas)* (Avelallemant and Held 1980, Hall and Shelton 1983) and *Tilapia* spp. e.g. *Tilapia aurea* was found successful in reducing TSS in Texas, USA (Stickney and Hesby 1978), *Tilapia nilotica* was being used to reduce TSS in tropical countries (Edwards and Sinchumpasak 1981). Also, silver carp (*Hypophthalmichthys molitrix*) and bighead carp (*Aristichthys nobilis*) were found to be capable of controlling plankton blooms in WSPs (Henderson 1978).

Using WSP effluent for agricultural land use is one of the most current creative measures investigated by researchers. This started with the worldwide public campaign to conserve resources, e.g. reuse and recycle. Seiber (1987) noted that effluent from well-designed WSPs can provide a prominent source of water for irrigation. Israel is one country which has found extended usage of pond effluent for agricultural irrigation and proved that effluents can yield more crop products (Shelet, Juanico and Vikinsky 1987).

However, Bartone and Arlosoroff (1987) stated that since there is an extensive use of sewage plant effluent for irrigation in several areas without adequate regard to the health aspect, it will cause damage rather than benefit to environment. This topic of reusing sewage effluent is still in the stage of investigation among current researchers in conjunction with environmental safety. The major sponsor of this research is the World Bank/UNDP.

Mara (1976) noted that WSPs needed minimal maintenance compared to other sewage treatment systems. He recommended that regular mowing of embankment grass and floating scum removal were both essential to WSP's system maintenance. Tariq and Ahmed (1981) observed that algal mats often appeared in the hot summer months. This is frequently found to be the most troublesome complaint from several operators of WSPs (Arthur 1981). Hess (1983), Tariq and Ahmed (1981) concluded that algal mats reduced a pond's performance and odour could be detected at a distance of about 15 m. The mats appeared to move by wind induction and in this way blooms are related to high temperature.

The number of operators handling a WSP system, according to Mara (1982), should be dependent on the size of the population served as shown in Table 3.5.

Table 3.5 The manpower requirement of a WSP plant.
(Source: Mara 1982)

Population served	Supervisors	Labourers
5,000	-	2
10,000	-	3
50,000	1	6
100,000	2	8

Other recommended data on staffing of a WSP plant can be drawn from Arthur (1983) indicated in Table 3.6.

Table 3.6 Recommended staffing numbers of WSP plant.
(Source: Arthur 1983)

Population served	10,000	25,000	50,000	100,000
Supervisor	-	-	1	1
Wastewater engineer	-	-	-	1
Laboratory technician	-	1	1	1
Assistant supervisor	-	1	2	2
Labourers	1	2	4	6
Driver	-	1	1	1
Watchman	1	1	1	3

According to Mara's (1982) publication in Table 3.5, the WSP plant servicing a population of 100,000 has a lower number of WSP operators than that of Arthur's (1983) in Table 3.6. These figures of WSP manpower will be mentioned in comparison to Khon Kaen WSP plant in Chapter 6. According to Ellis (1981), these operators should have duties as summarised:

(i) checking and cleaning preliminary processes,
(ii) cleaning and checking inlets and outlets,
(iii) cutting grass on banks,
(iv) checking banks for erosion,
(v) removing weed growth from shallow water
(vi) disposing detritus and screening,

(vii) noting the sludge accumulation in any part of the pond,

(viii) noting any evidence of short-circuiting.

The above responsibilities seem to be easier than those required for the conventional sewage plants, e.g. activated sludge (AS). The AS system requires a skilled operator and it also needs more sophisticated maintenance than WSPs.

In addition to these duties, Hess (1983) suggested that meteorological data such as air temperature, solar radiation, rainfall and wind should be recorded daily since these data would show the relationship of meteorological factors and pond performance. Physical and biological investigations are also essential for the operator, e.g. flow rates, water temperatures, water depths, sludge layer thicknesses for the physical aspects, and the examination of pH, BOD_5 ,COD, solids, and faecal coliforms form the basis of essential biochemical monitoring by these operators.

3.10 Construction, maintenance and operation costs of WSPs

Arceivala (1973) investigated the cost comparison of the WSP to aerated lagoon, oxidation ditch, and trickling filter and found that the WSP is the cheapest method of sewage treatment facility. Many authors have advocated WSPs as the most economical way of municipal wastewater treatment where land cost is not expensive. Examples were given by Culp (1961), Metzler and Culp (1961), Gloyna (1971), Mara (1976), Dinges (1982), Mara (1988). One of the recent cases for instance, was in Buffalo Town, Wyoming, USA. It was documented by Willey and Benjes (1985), that the proposed large scale WSP plant demonstrated a potential saving of up to 4 US million dollars compared to an oxidation ditch system.

Further, as referred to in the latest conference on WSP in Lisbon, Portugal 1989, the WSP has been viewed as the type of sewage treatment which can contribute to countries like Brazil, France, Venezuela, Portugal, Jordan and South Africa with the most cost-effective options compared to other systems (Hettiaratchi and Smith 1989).

Porges (1963) formerly surveyed 77 sewage treatment ponds in the USA, and found that construction and operating costs of WSP plants were $13.60 and $0.20-1.00 respectively. Calloway and Wagner (1966) revealed that construction cost of WSPs in USA is $14 per person and operating cost ranges from $0.20 to $1.00 per person per year.

Whereas in Africa, the costs of construction and operation are $5.50 and $0.42 per person per year respectively. Gloyna (1971) cited from Stanler and Meiring (1965) that annual running costs per capita for WSPs may be 1/2 to 1/10 of conventional sewage plants. Stoltenberg (1970) later found that a WSP's construction cost, in USA, is $43 per capita and maintenance plus operating cost is one-fifth of the conventional plant cost. From Table 3.7, Widmer (1981) demonstrated the ratio of construction cost of conventional treatment plants and WSP, such ratios show WSP can contribute a comparatively cheaper construction cost than the conventional systems.

Table 3.7 Ratio of construction cost of conventional treatment plants with respect to the cost of a pond treatment plant of the same capacity. (Source: Widmer 1981).

Design Population	ASP / WSP	TFP / WSP	PTP / WSP	ITP / WSP
100	3.7	4.5	3.2	4.0
1,000	4.1	4.6	3.4	2.8
10,000	4.8	4.8	3.8	2.0
100,000	5.8	5.0	4.2	1.4

Seabrook (1975) stated that the operating cost of Werribee farm in Melbourne, Australia, is $1.53 per capita in 1974, and after applying the plant's effluent to land irrigation for agriculture reuse, had a net return of $675,000. Ross, Boivin and Caverson (1984) surveyed operating cost comparisons of wastewater treatment plants in Canada and documented that secondary treatment devices such as activated sludge account for a cost per capita of $10, aerated lagoon $8 and WSPs $1. Further, they found that with SS and P removal, WSP is the cheapest method of wastewater treatment system.

In general, there is a great variation of construction costs in different regions at the same year basis such as in North America and Africa as Calloway and Wagner (1966) mentioned, this is mainly due to the difference of economic status in each region. The large increase of construction cost within the same country as compared in Porges (1963) and Stoltenberg (1970) resulted from the inflation rate in that country. However, many authors concluded that the WSP construction cost was lower than for other types of sewage treatment system. The lower operation cost of WSP resulted from less energy consumption.

For establishing a construction cost per capita model, several equations have been created. Reid and Muiga (1982) reviewed and analysed wastewater treatment cost models, such as the equations of Diachishin (1957), Logan *et al.* (1962), Butts and Evans (1970), Shah and Reid (1970) and Smith and Eiler (1970). It was found that most of these equations were based on statistics, particularly from linear regression and F-test. The application was expected to be valid in USA. Some of the models Reid and Muiga (1982) had investigated seemed to be inapplicable in practice. The author had tested some of these cost models and found that those above-mentioned equations could not be effectively applied to the Khon Kaen WSP cost. This was mainly from the difference in economic growth between one country and another.

However, the author had found that one of the current equations, which could be roughly applied to the Khon Kaen WSP, is by Hess (1981). This equation is the construction cost per capita model of WSP which can be shown in the following equation.

$$\text{Per capita cost (\$US)} = 0.1\,K\,(\text{Population})^{n-1} \qquad \text{equation 18}$$

where K = the coefficient of WSP types

n = the economic scale of WSP

This per capita cost equation of WSP has been investigated and found to be more reliable than any other model (Hess 1981). It is reliable in the sense that it achieved applicability by testing this model with historical plants in some countries e.g. India, Brazil and USA. Hess found that this model tended to be close to the real historical plant costs. However, this type of equation will be considered further in this study and will be used to analyse the Khon Kaen WSP construction cost in chapter 4.

Widmer (1981) cited from Gloyna (1979) analysed the energy requirement of WSPs compared to other systems. The results showed that WSPs required no energy inputs. Shown below is a comparison of proportional energy requirements using activated sludge system as a basis:

Activated Sludge Plant	1.00
Aerated Lagoon Plant	0.85
Rotating Biological Filter	0.13
Waste Stabilisation Ponds	0.00

It could be concluded that WSP is the most cost-effective sewage treatment system for developing countries, as many authors advocated and published, e.g. Hess (1983), Seiber (1987), and Pescod and Mara (1988).

There are many benefits of WSPs which can be summarised briefly as follows: (1) WSP offers distinctly low costs of construction and maintenance and low energy consumption, (2) WSP has a characteristic of ease in operation, thus skilled labour is not necessary, (3) WSP is viewed as a more flexible process than other systems since it can adapt to a wide fluctuation of liquid waste inflow, and (4) in terms of cost comparisons of purifying BOD_5, nitrogen and phosphorus, WSP was found to be superior to other systems. Thus, from these benefits, it is ascertained that WSP leads in minimising cost of sewage treatment facilities for communities in developing world.

The above review of literature on WSP systems acknowledges that WSP has a greater efficiency to purify wastewater and proves to be more economical to communities. A main concern of wastewater treatment is primarily to prevent health hazards to humans, and WSP can perform effectively to eradicate pathogens. Another reason for liquid waste purification is the prevention of eutrophication in receiving waterways. For this purpose, WSP also demonstrates effective reduction of nutrients such as nitrogen and phosphorus by algal uptaking, since that further minimises the degree of water stream pollution.

For the administrative feature, especially strategies to solve the problems of poor sanitation and community health in the developing world, World Health Organisation has fully advocated the WSP process as the most appropriate means of sewage treatment system in developing countries (Stevens 1970, Hess 1983).

However, WSP still requires further research centred on varying local conditions. Many engineering grounds, biochemical mechanisms and performance assessments of WSP need to be studied in different regions. This should also include a study of reuse of pond effluent for land applications such as in irrigation of crops. Current studies of WSPs are now approaching the search for mechanisms for improving effluent quality, especially SS content, in compliance with more stringent effluent standards. Proposed design formulae of WSPs and their consistent performance assessment are still being reported from several investigators based on specific locations. Land application and effluent reuses of WSP are also a major interest to some researchers, especially land application. Fish rearing in sewage ponds is also being studied in different countries.

CHAPTER 4

KHON KAEN WASTE STABILISATION POND SYSTEM

4.1 Khon Kaen city: background

Khon Kaen, one of 73 provinces in Thailand, is situated in the northeast of the country, 450 km from Bangkok. Khon Kaen is subdivided into 20 districts with a total area of 10,886 km^2 and a population of 1,851,106 (KKPO 1992). This province was established by the government as the capital of the Northeast. Khon Kaen is interconnected with the other 17 provinces of the northeast with linked routes to the north and the central parts of the country. This province then, is a city centre for regional government institutions, and socio-economic and academic establishments.

Khon Kaen city is the capital of Khon Kaen province, with a municipal area of 46 km^2 consisting of 4 subdistricts. The population of this municipality is 145,666 with a density of 3,167 persons per km^2. The city population has grown by 30,151 persons within a 10-year period (1983-1992).

The average rate in population growth of the city is around 3% per year resulting from immigration and natural growth rate. The annual per capita income in the city is still low, averaging 26,582 Bahts, equivalent to $A1,564. The economic growth rate of the city is considerable at 9.6% per year. The economic advancement is largely attributed to the expansion of banking, public service and commercial, retail and wholesale businesses (KKM 1991).

Khon Kaen city is facing environmental problems, the most critical among them are the problems of wastewater treatment and solid waste disposal (KKM 1986). The wastewater problem is mainly due to the sewage discharged from the city which eventually drains into the natural reservoirs, Bung Thung Sang, Nong Loeng Puai and Huy Phra Khu. The wastewater effluent from these reservoirs flows into the Pong and Chee rivers, the two main waterways in northeast Thailand, which supply water for human consumption and agricultural use.

The wastewater discharged from Khon Kaen city has adversely affected the people, animals and plants in the surrounding areas, around 5 million people in 4 provinces. With this adverse impact on the local environment, the WSP system was selected for the treatment of Khon Kaen's municipal sewage.

The Khon Kaen City Council, in conjunction with the central government, adopted the WSP project in 1986 with financial assistance from the World Bank. The Khon Kaen WSP plant was planned to be the central sewage treatment plant of Khon Kaen city.
Its construction began in 1987 and was completed in 1989. This plant, at present, serves only 70 percent of city households. This is because a portion of the city's sewerage system is still under construction and will not be completed until 1994. By then, the WSP will treat the whole city's sewage of the population of about 155,535.

One very interesting point to note about the Khon Kaen WSP is the fact that its physical structure is incomplete. The pumping house and the first two facultative ponds were finished according to the plan in 1989, but the three maturation ponds are only partly constructed. The defect of the three maturation ponds is that they are not lined with concrete slabs and their embankments are only made up of compact soil. This problem arose when the estimated cost of WSP construction in 1986 was insufficient to build the whole plant structure for implementation in 1987-1989. The main reason for this problem was that the Khon Kaen WSP structural design plan was modified from the original plan and this led to budget shortage in implementing the project.

The original Khon Kaen WSP structural plan, as designed by Thailand Institute of Scientific and Technological Research (TISTR), employed only one pond unit. Later, World Bank experts suggested the plan changed to a series of ponds. The emerging WSP plan was justified by World Bank personnel as being more beneficial than the former one for two reasons (1) it would provide the Khon Kaen WSP more flexibility in treating sewage, with less chance of short-circuiting and (2) more efficient sewage treatment could be achieved with the new plan (World Bank/UNDP 1986). These reasons for the World Bank changing the WSP plan are similar to those mentioned in Chapter 3 about the arrangement of the WSP.

Later, the World Bank and TISTR agreed to adopt a new WSP design which was finished in September 1986 (TISTR 1987). The increase in the number of ponds from one to five led to financial difficulties and the Khon Kaen City Council could not cope with the additional expense needed for completion of the WSP structure (KKCC 1987).

Nevertheless, the TISTR (1987), which is responsible for technical assistance in the design of the Khon Kaen WSP, felt that the absence of concrete lining on the three maturation ponds would not bear upon the WSP system. With reference to the TISTR statement, this study will determine whether such embankment lining is essential. The problems associated with whether ponds should be lined with concrete slabs or not will be discussed further in Chapter 6.

4.2 Khon Kaen wastewater profile

Khon Kaen's wastewater is largely generated by markets, restaurants, retail shops, offices and households. A typical feature of this city sewage is that it has received preliminary treatment in city households prior to being discharged to city sewers.

This sewage pre-treatment by households is required by the Health Law which defines that each household or institution must have a septic tank or seepage pit of its own in order to initially purify its sewage before discharging into public sewers. Consequently, the city sewage contains fewer pollutants when discharged to the sewage treatment plant, compared to cities in developed nations.

The quantity of sewage generated in this city has not been properly assessed. This also happens in other cities of Thailand which have little research on the quantity of sewage generation. It is normally found that the cities cannot supply such data publicly, and therefore they lack essential data for their sewage treatment planning. However, for Khon Kaen City, the estimated figure was obtained by TISTR (1986) from its survey prior to the construction of Khon Kaen WSP plant. TISTR documented that the daily sewage flow generated from the city is 6,740 m^3 per day in 1984, 10,000 m^3 in 1992 and expected to be 25,500 m^3 in 2001 (TISTR 1986a). TISTR (1986a) also noted that these figures of sewage flow tend to fluctuate due to (1) a high mobility of people in and out of the city and (2) it is difficult to quantify sewage flow precisely since the sewerage system has not been completed and most of the sewerage networks are scattered. Nevertheless, these figures, according to TISTR (1986a), can provide an 80 percent accuracy.

TISTR (1986a) also mentioned that Khon Kaen wastewater had considerably lower levels of impurities compared to the average figure for American cities. To illustrate the Khon Kaen wastewater characteristics, TISTR (1986a) analysed the Khon Kaen sewage prior to designing the WSP system. The values were obtained by the investigation of wastewater samples from four stations over the city, shown in Table 4.1.

Table 4.1 The Khon Kaen wastewater characteristics in 1986. (Source: TISTR 1986a)

Parameters	Station no. 1	Station no. 2	Station no 3	Station no 4
1.pH	7.3	7.4	7.4	7.7
2.DO mg/l	0	0.8	0	0.2
3.BOD_5 mg/l	22.6	25.3	23.6	20.3
4.TSS mg/l	24.3	58.0	37.6	21.3
5.Total N mg/l	13.81	16.55	15.18	12.6
6.PO_4-P mg/l	0.62	0.61	0.66	0.64
7.Faecal Coliform (*10^6 MPN/100 ml)	>2.4	>2.4	>2.4	>2.4

From Table 4.1 it can be seen that Khon Kaen City sewage has a BOD_5 level of 22.95 mg/L. According to Mara (1982), this level is low in terms of pollution since it is less than 200 mg/L of BOD_5. However, this BOD_5 figure could increase, in the case of Khon Kaen City, as a result of:

(1) the household pre-treatment system not working properly or with limited efficiency, then the amount of sewage discharged with a higher degree of pollution would increase. This is true according to this study since it was discovered that sewage inflow to the WSP has a high level of BOD_5 averaging 68 mg/L, more than that mentioned by TISTR (1986a). Referring to a discussion with a senior environmental scientist in the Department of Health, Thailand, this problem of high BOD_5 is due to a large extent to substandard pre-sewage treatment devices in households, especially in the new subdivisional areas. (Dr Taweesuk Panpeng pers. comm, Department of Health, Thailand 1993). Khon Kaen ranks third in the country for the number of commercial subdivisions and,

(2) socio-economic growth of the city. City life-style changes would result in higher consumption rates and thus wastewater characteristics would also change. Many publications have supported this concept, for instance, in Mara (1976, 1988), Metcalf & Eddy (1976) and Tchobanoglous (1991).

Table 4.1 also provides data to show that Khon Kaen sewage has a low value of DO, a relatively high content of nitrogen and a high coliform count.

Khon Kaen wastewater therefore has a high degree of pollution based on the levels of the three contaminants mentioned. A major impact will be the effect on aquatic animals. A low DO in the sewage will adversely affect the survival of fish and other organisms. Tchobanoglous (1991) published that a DO level less than 5 mg/L has an adverse effect on aquatic animals. Additionally the high faecal coliform would indicate that this type of wastewater is hazardous to community health.

The low values of BOD_5 shown in Table 4.1, TISTR (1986a) demanded a re-estimation of BOD_5 values for WSP design. The task of re-estimating Khon Kaen's wastewater characteristics was carried out by the Khon Kaen University (KKU 1986). The resulting new values of BOD_5 for WSPs are shown in Table 4.2.

It can be seen that the BOD_5 values in the two tables are totally different. This is because the Table 4.1 data was obtained from a study conducted in winter time and it used only 4 stations as sampling sites. In contrast, the values from the second study given in Table 4.2 were obtained during the hot season when the degree of decomposition of waste was very high and there were 39 sample sites (KKU 1986). However, the estimated quantity of sewage generated per day by the year 2001 was about the same for TISTR (1986a) and KKU (1986). TISTR estimated that sewage flow would be 25,500 m^3 per day while KKU estimated 25,495 m^3. Further, the latter estimated value of sewage generation in the city (25,495 m^3 by 2001) was adopted as the baseline data for the Khon Kaen WSP design (TISTR 1986b).

Finally, TISTR (1986b) accepted the values of BOD_5 determined by KKU (1986) for the Khon Kaen WSP design. The WSP was designed to operate until the year 2001, and the estimated value of BOD_5 found by KKU (1986) for that year was 110 mg/L and this will be used as the baseline data for sewage inflow to the WSP plant. But for safety reasons, the BOD_5 level of 110 mg/L was increased to 160 mg/L as the maximum value of BOD_5 that a WSP plant can handle from incoming sewage.

The BOD_5 value of 160 mg/L will be used in this study to check the consistency of the McGarry and Pescod (1970) model since this design was employed in this WSP plant design. The faecal coliform number used for the WSP design, TISTR (1986a) used the value in Table 4.1 which is $2.4*10^6$ MPN/100 ml. This value will also be taken into consideration in this study to analyse the predicted outcome of faecal coliform levels using the Marais (1974) model. These two points are intended to constitute the pre-assessment stage before the laboratory testings of WSP effluent. The laboratory results will be used for the post-evaluation of WSP performance.

Table 4.2. Values of BOD_5 in Khon Kaen municipal sewage.
(Source: KKU 1986)

Year	Sewage per capita	BOD_5 mg/L	BOD g/cap/ d	Sewage Quantity m^3 /d	BOD kg/d
1986	195	70	13.66	7,306	511
1991	201	79	15.97	11,520	910
1994	205	88	17.95	14,058	1,237
1996	208	93	19.43	19,295	1,794
2001	215	110	23.65	25,495	2,804

4.3 The Khon Kaen WSP plant

The Khon Kaen Wastewater Treatment Plant is located in southeast Khon Kaen, 3.5 km from the Khon Kaen city centre (see Figure 4.1). The plant receives city sewage through the main interceptor, the underground sewer. This WSP plant consists of two main components, the operation building located 1.8 km from the ponds, and the five sewage treatment ponds. The first two ponds are facultative ponds and the rest are maturation ponds connected in series. The first and the second facultative ponds have areas of 132,100 m^2 and 45,000 m^2, the three maturation ponds are 9,900, 3,600 and 7,200 m^2 respectively, and thus comprise a total area of 197,800 m^2.

The operation house, occupying 1,920 m^2, has two main functions:

(1) to receive sewage from the city via the main drainage interceptor and to pump it subsequently to the ponds and ,

(2) to act as the plant control building, which includes wastewater laboratory work and pump operation control.

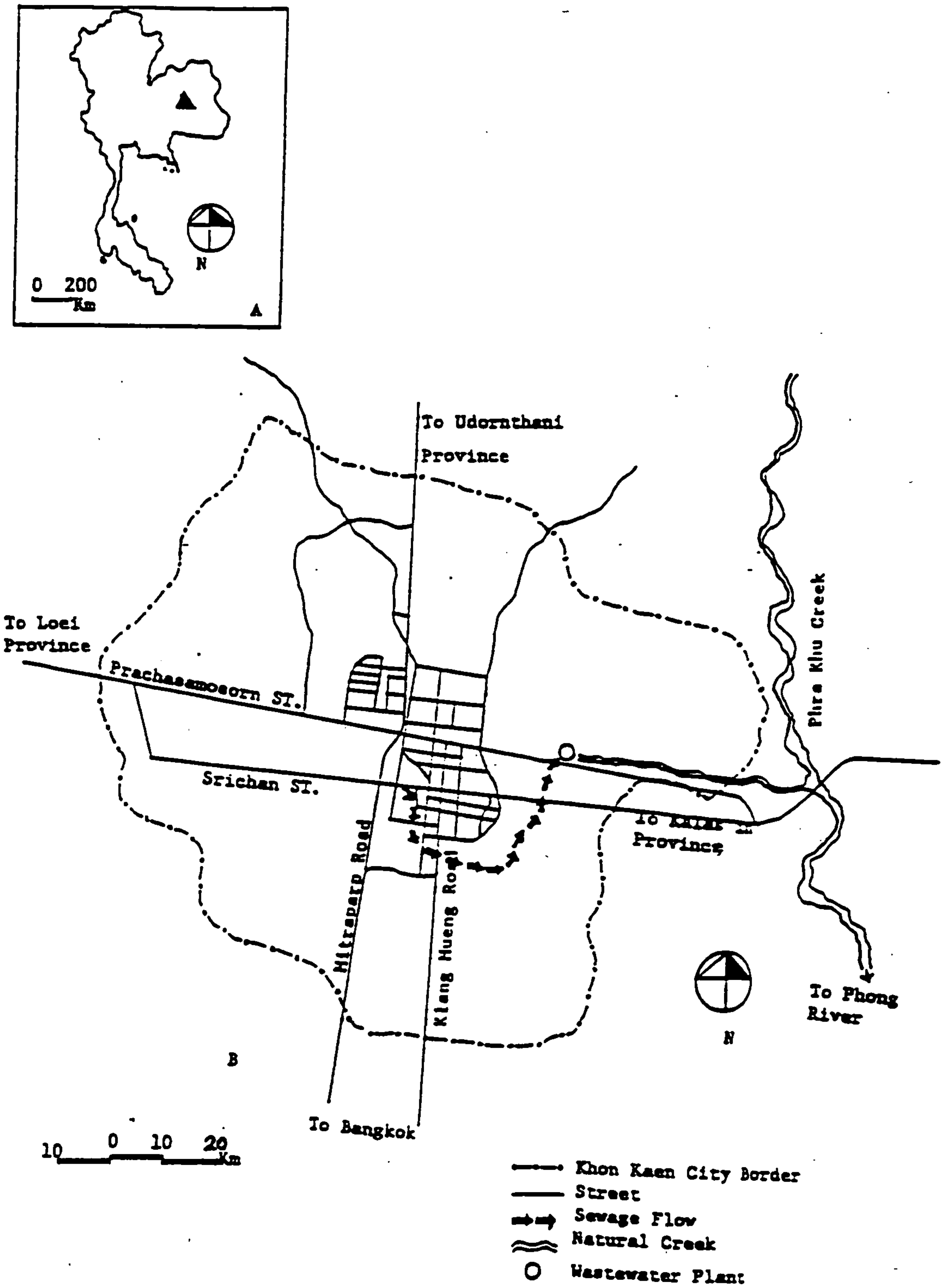
Inside the operation house there are two main compartments:

(1) the control building which functions as the main office building for operators running the plant. The automatic electrical switch panel for controlling the sewage pumps is located in this building, as well as the wastewater laboratory room and,

(2) the pumping well which has two main functions: (2.1) to act as a preliminary sewage treatment unit consisting of two compartments, bar screens, fine and course screens, and a grit chamber, and (2.2) to store and pump raw sewage to the pond system.

There are four submersible pumps in the station. The function of these pumps is mechanically controlled by the level of wastewater inflow in the pumping well and automatically regulated by the electrical switchboard in the control building.

Figure 4.1 The location of the Khon Kaen Wastewater Treatment Plant.
(Source: Adapted from Khon Kaen Municipality Map 1991)



The stabilisation pond is the most important unit in this sewage treatment plant. It has a total area of 228,800 m^2 (including roads and embankment). The first two facultative ponds serve to reduce BODs and the succeeding three maturation ponds serve to purify the water, removing harmful microorganisms. The operation of the five stabilisation ponds can be summarised as follows:

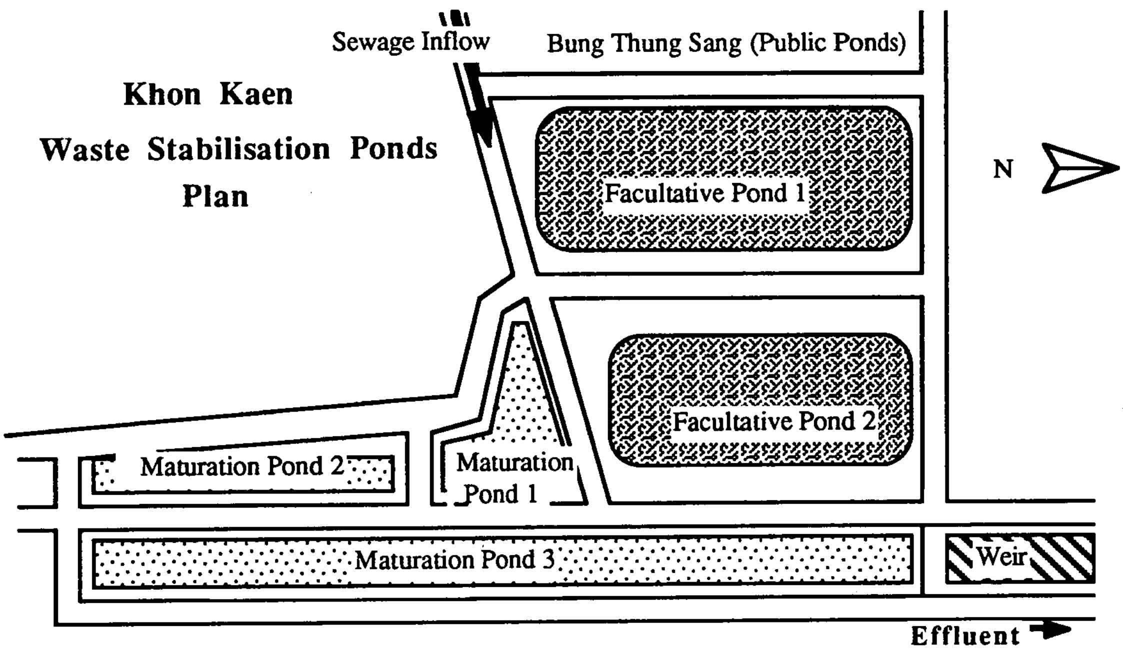
(a) the influent sewage enters the first facultative pond by passing through the distribution box. This box supplies wastewater to the first pond via the two outlets which are made of reinforced concrete pipes lying on the pond base,

(b) the wastewater is theoretically stored in the first pond for 10.36 days for purifying organic contents in the sewage. Afterwards the treated wastewater from the first pond flows into the second facultative pond through interconnected outlets (2 reinforced concrete pipes with a diameter of 1 m each), and is stored in this pond for 3.53 days,

(c) the wastewater from the two facultative ponds next enters the three maturation ponds through a rectangular weir, this weir also functions as a device for measuring daily wastewater flow within the plant, the theoretical retention times in the first, second and third maturation ponds are 0.78, 0.28 and 0.56 days respectively.

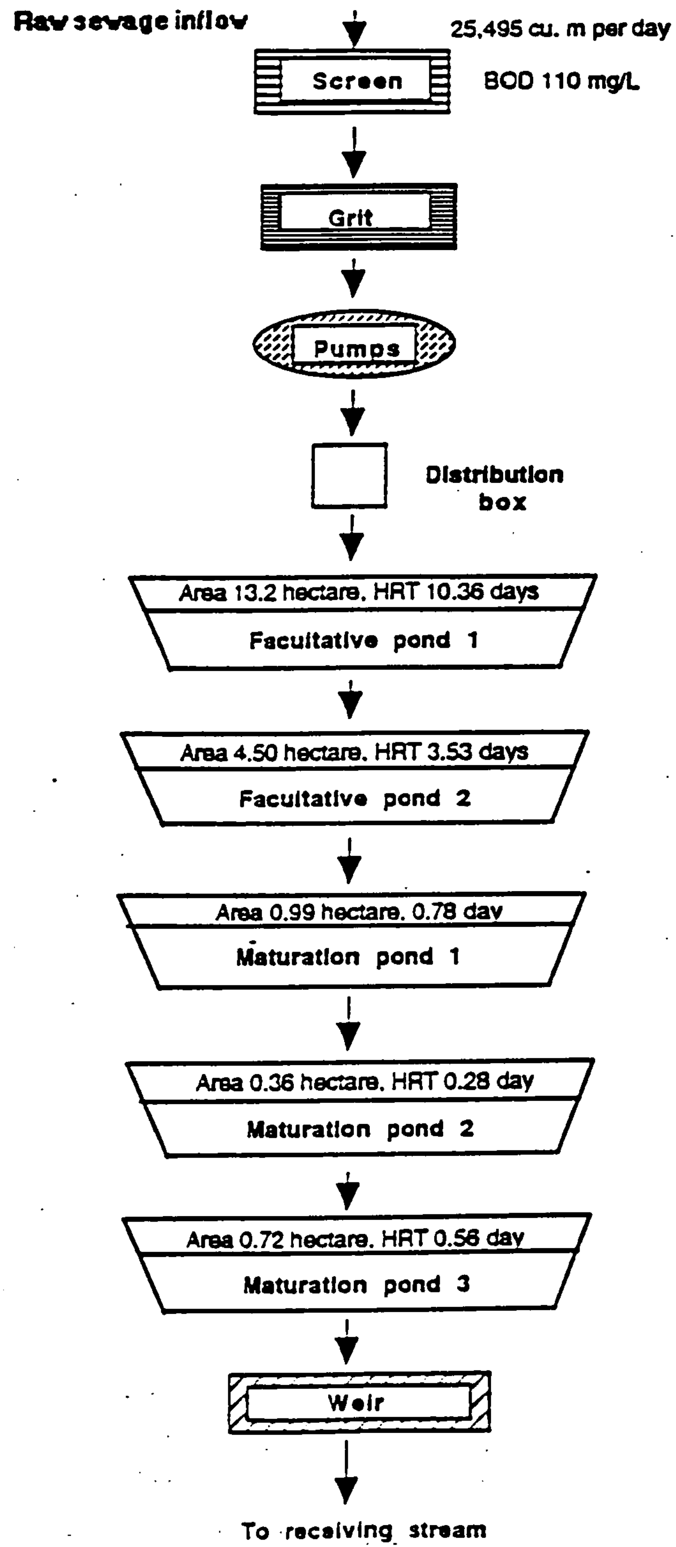
The ponds are rectangular in shape and connected in a series allowing wastewater flow throughout the system from the first facultative ponds to the last maturation pond (see Figures 4.2 and 4.3). The embankment of the ponds is made up of 95% compacted clay with a width of 4 m and a 1 in 2 slope. The pre-cast concrete slabs are placed on the embankment to inhibit vegetation growth, soil erosion and to prevent the breeding of mosquitoes in the banks. But, these slabs are limited to the first two facultative ponds, while the embankment of the three maturation ponds is covered with compact clay only. Each ponds' depth is 2 m and the base is lined with compact clay.

Figure 4.2 Plan of the pond including the 2 facultative and 3 maturation ponds.



(not drawn to scale)

Figure 4.3 The arrangement of the Khon Kaen WSP system, pond areas and theoretical hydraulic retention times (HRT).
(Source: TISTR 1986b)



4.4 Analysis of Khon Kaen WSP design models

There is no detailed information available about the design method of the Khon Kaen WSP plant. TISTR (1986b), as mentioned, employed McGarry and Pescod (1970) model for facultative ponds and Marais (1974) model for maturation ponds. It is necessary, at this stage, to trace back and analyse these two models in detail, in relation to the Khon Kaen WSP. Baseline data for the Khon Kaen WSP plant design will be considered, which includes projected data up to the year 2001. A review of these two formulae will be made, as well as other models mentioned in chapter 3. An analysis of the ponds' capacity will be made according to those models, using raw data for the year 2001. The full analysis of the Khon Kaen WSP plant is presented as follows:

(1) Baseline data for Khon Kaen WSP design (projected for 2001).

Sewage flow rate	25,495	m^3.
BOD_5 of incoming sewage	110	mg/L.
Influent faecal coliform numbers	$2.4 * 10^6$	MPN/100 ml.
BOD loading per day	2,804	kg
Mean temperature of the coldest month (for Khon Kaen city)	21.7	^{0}C (71.06 ^{0}F)
Latitude of Khon Kaen city	16	degrees N

Areas, volumes and retention times of each pond are summarised in Table 4.3. It can be seen that the first facultative pond is the biggest unit in the WSP system and the second maturation pond has the lowest volume.

Table 4.3 Areas, volumes and retention times of Khon Kaen WSP for the year 2001.
(Source: TISTR 1986b)

Ponds	Areas-m^2 (ha)	Volumes-m^3	Hydraulic retention times-d
Facultative pond I	132,100 (13.2)	264,200	10.36
Facultative pond II	45,000 (4.5)	90,000	3.5
Maturation pond I	9,900 (0.99)	19,800	0.78
Maturation pond II	3,600 (0.36)	7,200	0.28
Maturation pond III	7,200 (0.72)	14,400	0.56

This information will be used to calculate WSP design according to currently used models. Thus, some of the design models in Chapter 3 will be used in analysing the Khon Kaen WSP.

(2) There are six contemporary models which can be used to show the maximum organic loading value (BOD_5 kg/ha d) for facultative pond design. This can be applied to the Khon Kaen WSP. The equations are given below:

(2.1) McGarry and Pescod model (1970):

$$L_m = 11.2\ (1.054)^T \qquad \text{(equation 7)}$$

where L_m = minimum BOD_5 loading (kg/ha d)
T = ambient mean monthly temperature ^0F (Khon Kaen city = 28.3 ^0C = 82.9 ^0F)

therefore
$$L_m = 11.2\ (1.054)^{82.9}$$
$$L_m = 876.40 \quad \text{kg/ha d.}$$

Tam (1982) later modified this formula to include a safety factor of 1.5. Given this, the value of the maximum BOD_5 loading of the Khon Kaen WSP is therefore, 876.40/1.5 = 584.27 kg/ha d.

(2.2) Mara model A (1976) is a modification of the McGarry and Pescod model (1970):

$$L_m = 7.5\ (1.054)^T \qquad \text{(equation 9)}$$

therefore
$$L_m = 7.5\ (1.054)^{82.9}$$
$$L_m = 586.88 \quad \text{kg/ha d.}$$

(2.3) Mara model B (1976):

$$L_a = 20T - 120 \qquad \text{(equation 10)}$$

where L_a = maximum BOD_5 loading (kg/ha d)
T = mean local temperature of the coldest month (Khon Kaen = 21.7 ^0C)

therefore $L_a = 20\,(21.7) - 120$

$= 314$ kg/ha d

(2.4) Arceivala model (1970):

$$L_a = 375 - 6.25\,L \qquad \text{(equation 12)}$$

where L_a = maximum BOD_5 loading kg/ha d

L = latitude (valid between $8\,^0$ - $36\,^0$N)

(Khon Kaen city latitude = $16\,^0$N)

therefore $L_a = 375 - 6.25{*}16$

$= 275$ kg/ha d.

(2.5) The Asian Institute of Technology (AIT) model (Ahmed 1981) is also a modification of McGarry and Pescod model (1970):

$$L_m = 8\,(1.054)^T \qquad \text{(equation 13)}$$

therefore $L_m = 8\,(1.054)^{82.9}$

$L_m = 626$ kg/ha d

(2.6) Arthur (1983) model is a refinement of the Mara model (1976):

$$L_a = 20T - 60 \qquad \text{(equation 14)}$$

therefore $L_a = 20\,(21.7) - 60$

$L_a = 374$ kg/ha d

The areal organic loading formulae show that the permissible BOD_5 loading values of the Khon Kaen WSP vary according to the seven models. The AIT model provided the highest figure for allowable BOD_5 loading ie 626 kg/ha d, while the Arceivala model gave the least amount of BOD_5 loading of 275 kg/ha d.

The estimated values for the year 2001 are 2,804 BOD_5 loading (kg per day) with the area of the two facultative ponds being 17.7 ha. Therefore the BOD_5 loading of the FPs is only 2,804/17.7 or 158.4 kg/ha d. Thus, to sum up, these figures, by not exceeding the values calculated, are acceptable in terms of the above models.

(3) Some design models relate to the prediction of BOD_5 removal. Two of these attempt to indicate the amount of BOD_5 removal per unit area (BOD_5 kg/ha d). These are shown below and can be applied to the Khon Kaen WSP design.

(3.1) McGarry and Pescod model (1970):

$$\text{hydraulic} = 10.75 + 0.725\ L_i \qquad \text{(equation 8)}$$

where L_r = BOD_5 removal per unit area (kg/ha d)
L_i = inflow BOD_5 loading (kg/ha d).

(3.2) Yanez model (1980):

$$L_r = 0.8193L_i - 7.81 \qquad \text{(equation 11)}$$

where L_r = BOD_5 removal per unit area (kg/ha d)
L_i = inflow BOD_5 loading (kg/ha d).

A comparison can be made between equations 8 and 11. For the Khon Kaen WSP system, by applying one value of permissible inflow BOD_5 loading, for example, 314 kg/ha d (Mara 1976), then using equation 8 the maximum BOD_5 removal per unit area (L_r) will be 238.4 kg/ha d. If equation 11 is used, then the maximum BOD_5 removal value will be 249.4 kg/ha d. The result showed that the Yanez model (1980) provides a slightly higher value than that of McGarry and Pescod (1970).

However, since the estimated value for inflow is 158.4 kg/ha d, by 2001, then using equation 8 and 11 the BOD_5 removal values are 125.6 and 121.9 kg/ha d respectively. These two values, therefore, are below the maximum permissible values computed by using McGarry and Pescod (1970) and Yanez (1980) equations as mentioned above.

Equations 8 and 11 reveal a point that requires further investigation. According to these two models, the BOD_5 effluent levels of the facultative ponds can be predicted and can achieve the expected outcome of wastewater quality from facultative ponds during the design period. The prediction of BOD_5 levels of the Khon Kaen WSP effluent, in this study, will use equations 8 and 11 and take FP I and FP II as one unit area of FP for calculation.

This analysis aims to compare both equations, to determine whether they contribute similar expected values of BOD_5 effluent or not. The results from these two equations can be seen in Tables 4.4 and 4.5.

Table 4.4, using equation 8 shows the theoretical values of BOD_5 in the effluent from the FP I and II. It can be seen that the final outflow from FPs is 22.8 mg/L and is close to the national effluent standard of 20 mg/L.

For equation 11, Table 4.5 shows that the results of BOD_5 in the effluent from FP I and FP II are not much different from those of equation 8 in Table 4.4.

However, these numbers are, in theory, only estimated values projected for the year 2001. In 1993, the values of inflow BOD_5 loadings, BOD_5 removals and retention times were different from values expected in 2001. It is worthwhile to analyse the expected outcome of BOD_5 mg/L from the current actual flow rates and BOD_5 content of raw sewage. This latter exercise will be presented in topic 4.5.

Table 4.4 The expected levels of BOD_5 effluent from the two facultative ponds of the Khon Kaen WSP sewage plant using the McGarry and Pescod model (1970).

Data	Facultative pond I and II
1 BOD_5 loading	2804 kg/d
2 Area	17.7 ha (177100 m^2)
3 Pond depth	2 m
4 HRT	13.9 d
5 Inflow BOD_5	158.4 kg/ha d (a)
6 BOD removal (equation 8)	125.6 kg/ha d (b)
7 BOD_5 effluent	$\frac{(a-b)*17.7*13.9*1000}{177100*2}$ = **22.8 mg/L**

Table 4.5 The expected levels of BOD_5 effluent from the two facultative ponds of the Khon Kaen WSP sewage plant using the Yanez model (1980).

Data	Facultative pond I and II
1 BOD_5 loading	2804 kg/d
2 Area	17.7 ha (177100 m^2)
3 Pond depth	2 m
4 HRT	13.9 d
5 Inflow BOD_5	158.4 kg/ha d (a)
6 BOD removal (equation 11)	121.9 kg/ha d (b)
7 BOD_5 effluent	$\frac{(a-b)*17.7*13.9*1000}{177100*2}$ = **25.4 mg/L**

In addition to the two models providing the BOD_5 effluent values in (3) above, there is another group of equations which are valuable for investigating the consistency of (3), as shown in (4).

(4) Equations for facultative pond design related to retention times and depths are as follows:

(4.1) Mara (1976) equation:

$$\frac{L_e}{L_i} = \frac{1}{1 + kt^*} \qquad \text{(equation 3)}$$

where L_e and L_i = the values of BOD_5 (mg/L) for effluent and influent respectively

t^* = the mean retention time in days

k = the first order rate of constant BOD removal ($0.23\ d^{-1}$)

(4.2) Marais and Shaw formula A (1961):

$$L_e = \frac{600}{(0.18d + 8)} \qquad \text{(equation 4)}$$

where L_e = effluent BOD_5 (mg/L)

d = pond depth

(4.3) Marais and Shaw formula B (1961):

$$L_e = \frac{L_i}{(0.17\,R + 1)} \qquad \text{(equation 5)}$$

where L_e = effluent BOD_5 (mg/L)

L_i = influent BOD_5 (mg/L)

R = retention time

Using equations 3 to 5 for the data of Khon Kaen wastewater in 2001, values of effluent BOD_5 from facultative ponds (combine FP I and II in one unit area) are shown in Table 4.6.

Table 4.6 The BOD_5 effluent of the Khon Kaen WSP for 2001 with reference to the models of Mara (1976) and Marais and Shaw A, B (1961).

Ponds	Mara (1976) model (effluent BOD_5 mg/L)	Marais and Shaw (1961) model A (effluent BOD_5 mg/L)	Marais and Shaw (1961) model B (effluent BOD_5 mg/L)
Facultative pond I and II	26.2	71.7	32.7

From Table 4.6, it is obvious that the values of BOD_5 effluent from the two FPs are higher than those of McGarry and Pescod (1970) and Yanez (1980) which are found in Tables 4.4 and 4.5 respectively. All of the calculated BOD_5 levels in Tables 4.4, 4.5 and 4.6 are higher than the standard value-20 mg/L, so the subsequent ponds, MPs, will be expected to have a role in reducing BOD_5 meeting the effluent standard.

Unlike Mara (1976) and Marais and Shaw (1961) model B, the Marais and Shaw (1961) model A provides a design suitable for one big pond unit. In this case, the BOD_5 effluent result is not applicable to a WSP plant if it is laid out in a series of ponds.

A comparison of the BOD_5 effluent values of both FPs using equations 3, 5, 8 and 11 as shown in Tables 4.4, 4.5 and 4.6 should be made.

(a) the McGarry and Pescod (1970) formula has the value of BOD_5 effluent as 22.8 mg/L. This value is approximately the same range as Yanez (1980) and Mara (1976) which are 25.4 mg/L and 26.2 mg/L respectively.

(b) the Marais and Shaw equation (1961) gave the highest value of BOD_5 effluent, that is 32.7 mg/L.

(5) The last of the design formulae considered here is the maturation pond design. This design model is principally concerned with bacteria reduction. The only design model used is that of Marais (1974). Marais used the faecal coliform counts as an indicator of the WSP bacterial treatment performance. As bacterial reduction also occurs in facultative ponds all pond units will be considered as faecal coliform reducers in accordance with the Marais model.

(5.1) Marais formula (1974):

$$N_e = \frac{N_i}{(1 + k_b t)} \qquad \text{(equation 15)}$$

where

N_e = counts of FC / 100 ml in effluent

N_i = counts of FC / 100 ml in influent

k_b = first order rate constant for FC removal d^{-1}

= $2.6\ (1.19)^{T-20}$

= $2.6\ (1.19)^{21.7-20}$

= $3.49\ d^{-1}$

t = retention time-d.

(5.2) For a series of ponds like the Khon Kaen WSP, then equation 15 should be changed as follows:

$$N_e = \frac{N_i}{(1 + k_b t_1)\ (1 + k_b t_2) \ldots (1 + k_b t_n)} \qquad \text{(equation 16)}$$

Data (1) giving the initial count of faecal coliform entering the Khon Kaen WSP system from (1) is $2.4*10^6$ MPN/100 ml, so the effluent from the last pond can be calculated with equation (5.2):

$$N_e = \frac{2400000}{(1+3.49*10.36)(1+3.49*3.5)(1+3.49*0.78)(1+3.49*0.28)(1+3.49*0.56)}$$

N_e = 173 MPN/100 ml.

The faecal coliform count in the final effluent of Khon Kaen WSP is 173 MPN/100 ml. This falls below the standard for faecal coliform counts in the effluent of a WSP which is <5000 MPN/100 ml (Mara 1976). The current standard in Thailand for faecal coliform (FC) in effluent is limited to 1,000 MPN /100 ml, as shown in Table 2.5.

Therefore the final effluent of the Khon Kaen WSP, in terms of FC content, still meets the Thai standard.

In summary, by using data for the year 2001 in (1), it can be seen that the final effluent quality of BOD_5 from the FP ponds, in theory using the current design models, is in general not able to meet the country's effluent standard, the proceeding MPs expected to have an additional role in BOD_5 reduction to meet the effluent standard. Also by using the Marais (1961) model, theoretically, the final effluent from the last pond contains an FC count less than that defined by the Thai standard.

Since the conventionally designed WSP methods focus only on BOD_5 and FC values, at present there is no formula to apply to other contaminant reductions such as TSS, phosphate and nitrogen. Pescod and Mara (1988) recommended that it is necessary for any country planning to develop WSPs, to investigate models aimed at achieving reductions in these three contaminants.

Currently, there are many studies being carried out in different global regions, looking at the performance of WSPs. The focus is on the efficiency of the pond system or biological mechanisms within it. The aim is to obtain information on pond performance associated with contaminant reduction which is suited to specific local areas.

4.5 Theoretical results based on current Khon Kaen sewage inflow data

In this section, the aim is to present predicted results from theoretical bases by applying current data (1993) obtained in this study. The baseline data inputs in this section are different from the data used in section 4.4, which were projections for the year 2001. However, the mean temperature of the coldest month is the same, since this information was gained from historical records. Data for quality of influent raw sewage for the Khon Kaen WSP, are investigated in this study over four months, as shown in Tables 4.7.

Table 4.7 The characteristics of Khon Kaen's raw sewage prior to entering the Khon Kaen WSP plant.

Current sewage characteristics	Values
1. Sewage inflow	7914.25 m^3/d
2. BOD_5 inlet	68 mg/L
3. Faecal coliform inlet	$2.65*10^7$ MPN/100 ml
4. BOD_5 loading	534 kg/d
5. Mean temperature of the coldest month	21.7 ^0C

According to average actual BOD_5 loading at present it is found that BOD_5 loading to the FPs is only 534 kg/d (from Table 4.7), that is 534/17.7 = 30.2 kg/ha d. This figure is much lower than maximum values mentioned in (2.1) to (2.6), which provide the greater values of 876.4 kg/ha d (2.1) to 275.0 kg/ha d (2.6). Thus the current BOD_5 loading to the two FPs does not exceed the values calculated in those FP design models [(2.1) to (2.6)].

Table 4.7 shows that the quantity of raw sewage inflow to this WSP plant is currently 7,914.25 m^3, approximately three times less than that predicted for 2001 (25,495 m^3). Further, the influent BOD_5 (68 mg/L) to the pond is approximately two times less than that the value estimated for 2001 (110 mg/L). The only contaminant which presently has a greater value than that for the year 2001 is the faecal coliform count which averages $2.65*10^7$ MPN/100 ml.

Table 4.8 The hydraulic retention time of each pond in the Khon Kaen WSP plant.

Ponds	Hydraulic retention times (days)
1. Facultative pond I	33.38
2. Facultative pond II	11.37
3. Maturation pond I	2.50
4. Maturation pond II	0.91
5. Maturation pond III	1.82

Current hydraulic retention time (HRT) is another determinant that varies greatly from that of the predicted HRT for the year 2001. The total current HRT is averaged at 49 days, whereas for the year 2001, HRT is estimated to be 15 days.

Later, in this study, in analysing the design model equations, a prediction of BOD_5 outflow from the FPs and faecal coliform numbers in the final effluent from the last pond will be made. In contrast to section 4.4, this analysis will use data from the current figures from the Khon Kaen WSP as shown in Tables 4.7 and 4.8. Equations 3, 4, 5, 8, 11 and 16 will be reused for the analysis.

Equations for BOD_5 removal per unit area (equations 8 and 11) are firstly used to present the predicted values of BOD_5 effluent. Secondly, another group of equations for predicting BOD_5 effluent (equations 3, 4 and 5) are used in order to compare the values obtained with equations 8 and 11. Lastly, equation 16 will be used for predicting the FC count in the final effluent.

Equation 8 cannot predict the BOD_5 value of the outflow from the FPs. This is because equation 8 gives the level of BOD_5 removal per unit area (32.6 kg/ha d) exceeding the influent BOD_5 (30.2 kg/ha d). However, it can assume that the effluent BOD_5 level from the FPs will be at a comparatively low level as referred to in this equation.

When applying Yanez model (equation 11), the resulting BOD_5 effluent is shown in Table 4.9. The BOD_5 effluent expected from this equation is 29.7 mg/L and that will certainly be higher than the value of equation 8.

However, the expected BOD_5 levels in Table 4.9 will be compared with the laboratory results obtained for BOD_5 levels from the FPs effluents. The matter of any difference between theoretical and actual BOD_5 will be discussed in Chapter 6.

Table 4.9 The expected levels of BOD_5 effluent from the two facultative ponds of Khon Kaen WSP plant, using Yanez equation (1980).

Data	Facultative pond I and II
1 BOD5 loading	534 kg/d
2 Area	17.7 ha (177100 m^2)
3 Pond depth	2 m
4 HRT	44.7 d
5 Inflow BOD5	30.2 kg/ha d (a)
6 BOD removal (equation 11)	16.9 kg/ha d (b)
7 BOD5 effluent	$\frac{(a-b)*17.7*13.9*1000}{177100*2}$ = **29.7 mg/L**

Another approach used to determine the BOD_5 levels in outflow from FPs is derived from equations 3, 4 and 5. Table 4.10 shows expected values of BOD_5 from FP effluent. It is found that Mara's equation (1976) gives approximately the same result as that of Marais and Shaw model B (1961). But, Marais and Shaw model A (1961) shows a different value.

Table 4.10 Values of BOD_5 effluent from FPs calculated with the models of Mara (1976) and Marais and Shaw A, B (1961).

Ponds	Mara (1976) model (effluent BOD5 mg/L)	Marais and Shaw (1961) model A (effluent BOD5 mg/L)	Marais and Shaw (1961) model B (effluent BOD5 mg/L)
Facultative pond I and II	4.1	71.7	7.9

Comparing BOD_5 values in Tables 4.9 and 4.10 the Marais and Shaw model A (1961) provides the highest figure for BOD_5 levels (71.7 mg/L). Marais and Shaw model B (1961) gives a figure for BOD_5 (7.9 mg/L) similar to that of the Mara equation (1976) (4.1 mg/L). Yanez equation provides a BOD_5 value (29.7 mg/L) greater than that of Mara (1976) and Marais and Shaw (1961).

With reference to BOD_5 values conforming to the national standard (not exceeding 20 mg/L), Mara equation (1976) and Marais and Shaw model B (1961) show that the effluent values of BOD_5 from FPs can meet the standard, while Yanez equation (1980) shows values to be slightly over the standard.

The above description gives a theoretical picture of expected BOD_5 values at the Khon Kaen WSP plant, using current data for raw sewage. In fact, any value calculated from those equations is expected as a general idea of effluent BOD_5 values from the FPs. The effluent BOD_5 values derived from such equations would be only a general guide to the designer to gain an ideal picture of BOD_5 effluent from the FPs. In this study, the investigation of such equations serves to look at which equation can provide the closest value to the actual result.

Another concern is the bacteria reduction which can be determined by examining data of bacteria entering the plant. From laboratory testing, faecal coliform (FC) in the raw sewage averages $2.65*10^7$ MPN/100 ml.

According to equation 16 the faecal coliform count in the final effluent from the plant can be determined as follows:

$$N_e = \frac{26500000}{(1+3.49*33.38)(1+3.49*11.37)(1+3.49*2.5)(1+3.49*0.91)(1+3.49*1.82)}$$

$$N_e = 0.058 \text{ MPN/100 ml.}$$

It can be seen that the number of FC is dramatically reduced from $2.65*10^7$ (in raw sewage)(Table 4.7) to 0.058 MPN/100 ml (in the final effluent of the last pond). This theoretical value, however, appears to be impracticable, due to a divided fraction in this formula which involves HRT. However, such a value can provide a promise that this plant can theoretically reduce the FC number and the final FC can meet the standard. Predicted values of BOD_5 in the effluent from FPs and FC number from the last pond will be further investigated by comparing these values with the laboratory results in Chapter 6.

4.6 Population served, costs and maintenance of the plant

The Khon Kaen WSP, being the central sewage treatment system, currently serves a population of 109,241 from 21,007 households in Khon Kaen municipality (KMO 1993). The total construction cost of the plant in 1989 was $A 0.85 million (15.3 million bahts). Thus, this plant costs $A 7.78 per capita or $A 40.46 per household.

Equation 18 (Hess 1981) in Chapter 3 gives a calculation of construction cost per capita of WSP. The Khon Kaen WSP construction cost per capita can be estimated according to this equation as follows:

The construction cost per capita of WSP ($US)		=	0.1 k (population)$^{n-1}$ (equation 18)
where (for a series of WSP)	k	=	1334
	n	=	0.717
therefore the Khon Kaen WSP cost		=	0.1*1334*(109241)$^{-0.283}$
		=	5 ($US)
		=	7.35 ($A)

This $A7.35 is the cost from 1981 and adjusted to 1989 costs when considering the inflation rate, the value will be $A9.29. It can be seen from Hess's equation (1981) that the calculated cost of WSP is considerably different from the real cost of Khon Kaen WSP. This equation was found to be effective in USA but varied in Khon Kaen, because the inflation rate of Thailand is high in general and Thailand had a the higher economic growth rate in the 1980s compared to USA. However, this equation gives the value closest to the actual Khon Kaen WSP plant construction cost than the other formulae.

Maintenance costs of the Khon Kaen WSP plant range from $A 3,582 in 1989 to $A 27,678 in 1992 as shown in Table 4.11. The greatest expense at all times is due to power costs. The per capita cost per year for maintenance ranged from $A 0.033 in 1989 to $A 0.25 in 1992.

Table 4.11 Maintenance costs of the Khon Kaen WSP plant since starting operation in 1989.
(Source: KKM 1993)

Details	1989	1990	1991	1992
1.Equipment repair costs ($A)	833	3112	1451	5788
2.Power costs ($A)	2644	9550	15340	20199
3.Office supply costs($A)	105	77	1443	1691
4.Total ($A)	3582	12739	18234	27678

There are presently five full-time plant operators working in the plant. One is the supervisor and the rest are labourers. The assigned jobs for plant operators are: controlling the pumps, mowing weeds, removing algal mats from the ponds and inspecting the plant.

Laboratory equipment is provided for testing the plant effluent but at present this is not being carried out. Pumping is presently controlled manually by operators, due to the breakdown of the automatic electrical switchboard which was designed to control the function of the pumps.

CHAPTER 5

MATERIALS AND METHODS

5.1 General considerations

In this study, investigation methods are used covering aspects of biological performance and management of the Khon Kaen WSP sewage plant. These include:

(1) physical, biochemical and bacterial tests to evaluate Khon Kaen WSP performance,
(2) identification of WSP biota species existing in the ponds,
(3) comparative study of actual and theoretical results of Khon Kaen WSP performance,
(4) study of expenditure for Khon Kaen WSP in terms of construction, operation and maintenance costs as well as manpower contribution to the plant, and
(5) other studies involving operation, maintenance and trouble shooting in the plant.

Firstly, this study aims to evaluate WSP efficiency in compliance with Thai effluent standards, and secondly aims to investigate appropriate WSP performance in reducing other major contaminants. Though not mentioned in the country's standard, they have a major role in polluting the receiving waterway.

The initial tests provided by the Thai national standards are for BOD_5, TSS, TDS, pH total and faecal coliform count. Though COD, temperature, DO, NH_3-N, NO_3-N, NO_2-N, and PO_4-P are not specified in the country's standard, this study will include these parameters, since this latter group of tests will allow this study to attain the second aim.

Apart from these physical, biochemical and bacterial tests, the other environmental factor to be measured is light intensity which is expected to influence biooxidation reaction inside the pond. This measurement will be used to determine any correlation between light intensity, DO and temperature within the ponds. This test also contributes to the attainment of the second objective of the study.

However with limitation of time and resources, the tests for light intensity, DO and temperature were conducted for only a short period of time and were carried out on a different period from the above tests. The complete list of parameters tested in this study are shown in Table 5.1.

Table 5.1 summarises the wastewater parameters and methods of examination used in the Khon Kaen WSP Wastewater Plant study. The examination methods employed are taken from the procedures in Standard Methods for the Examination of Water and Wastewater, 17th edition, American Public Health Association (APHA), Washington DC., 1989.

Table 5.1 Parameters investigated and procedures used in this study.

Parameters	Units	Sampling Sites (see Figure 5.1)	Methods & Apparatus
1.Biochemical Oxygen Demand (BOD)	mg/l	A, B, C, D, E and F	Azide Modification Method
2.Chemical Oxygen Demand (COD)	mg/l	A, B, C, D, E and F	Dichromate Reflux Method
3.Total Suspended Solid (TSS) (Nonfiltrable Residue-NFR)	mg/l	A, B, C, D, E and F	Uses the method illustrated in The Standard Method of APHA-1989
4.Total Dissolved Solid (TDS) (Filtrable Residue)	mg/l	A, B, C, D, E and F	Uses the method illustrated in The Standard Method of APHA-1989
5.pH	pH value	A, B, C, D, E and F	Electrometric Method using pH Meter Ionanalyzer (ORION Model EA 940™)
6.Dissolved Oxygen (DO)	mg/l	A, B, C, D, E and F	Azide Modification
7.Phosphate Orthophosphate (PO_4-P)	mg/l	A, B, C, D, E and F	Photometer Method using Photometer (PALIN TEST)
8.Nitrate (NO_3-N)	mg/l	A, B, C, D, E and F	Electrometric Method using Ionanalyzer (ORION Model EA 940™).
9.Nitrite (NO_2-N)	mg/l	A, B, C, D, E and F	Electrometric Method using Ionanalyzer (ORION Model EA 940™).

10.Ammonia (NH_3-N)	mg/l	A, B, C, D, E and F	Indophenol Method using test kit VISOCOLO-DEV.
11.Total Coliform Bacteria	MPN/100 ml	A, C and F	Multiple-Tube Fermentation Technique.
12.Faecal Coliform Bacteria	MPN/100 ml	A, C and F	Multiple-Tube Fermentation Technique.
13.Temperature	Degree Celsius	A, C and F	Thermometer.

5.2 Biota species examination

Since fish species, algae and other aquatic plants have a purification function in wastewater treatment of the WSP, it is valuable to have those biota identified. Another reason is that species identification will draw a comprehensive understanding of the pond ecosystem. For instance, planktonic microorganisms are responsible for purifying phosphorus and nitrate compounds. This investigation will use a general approach which will specify the function of algal species in WSPs. Fish inhabiting WSPs, which in general have an important role in SS reduction, will also be identified. Aquatic plant species flourishing inside WSPs, which have a prime role in nutrient reduction, will also be described.

Since algal species require more skill in identification, the biology laboratory and staff from the Faculty of Science, Khon Kaen University were requested to assist in the phytoplankton investigation. Equipment used for algal species identification was a binocular microscope. The sampling sites used for collection of algal samples were the same sites as those used for biological and bacteriological tests (as recommended in Standard Methods for the Examination of Water and Wastewater, APHA, 1985).

The method employed in the collection of liquid wastes at the 0.5 m subsurface level was a 2-litre polyethylene bottle. A compound microscope was used for identification of planktonic species.

Pisciculture taking place in the WSPs will be recorded since residents living near the plant regularly catch fish for their consumption. Fish species were identified from this harvest. Aquatic plants which are growing near the embankment or extending into the WSPs were also identified.

5.3 Testing WSP performance in relation to conventional design formulae

There are several equations which apply to the design of a WSP system, especially involving WSP effluent prediction figures. This study aims to test whether the Khon Kaen WSP system operates in accordance with some of those equations mentioned in Chapter 4. The conventional equations relating to BOD_5 effluent required to be tested are the design models in equation 3 of Mara (1976), equations 4 and 5 of Marais and Shaw models A and B (1961), equation 8 of McGarry and Pescod (1970) and equation 11 of Yanez (1980). All of these design equations predict the levels of BOD_5 which reflect the WSP treatment efficiency. The actual figures from laboratory results of BOD_5 levels in the WSP effluent will be compared to the calculated values expected from the five equations. Equation 16, which predicts the outcome of FC numbers from the WSP plant will also be examined by comparing it to the bacteriological tests carried out as illustrated in Table 5.1.

5.4 Plant construction, operation and maintenance expenses

The Khon Kaen WSP construction, operation and maintenance costs will be taken into consideration since these costs can provide a cost-benefit figure for a community-applied WSP system. Expenses of the plant will be taken from data recorded by the Khon Kaen City Council. Expenditure on record, noted in Baht (Thai currency system), is converted into equivalent Australian dollars. Cost per capita will be used for computing construction, operation and maintenance expenses.

5.5 Trouble shooting in the operation and maintenance of the plant

An observation on operation and maintenance of the plant during the time of this study was made. The historical performance of the plant will also be presented as well as information from discussions with personnel who are operating the plant.

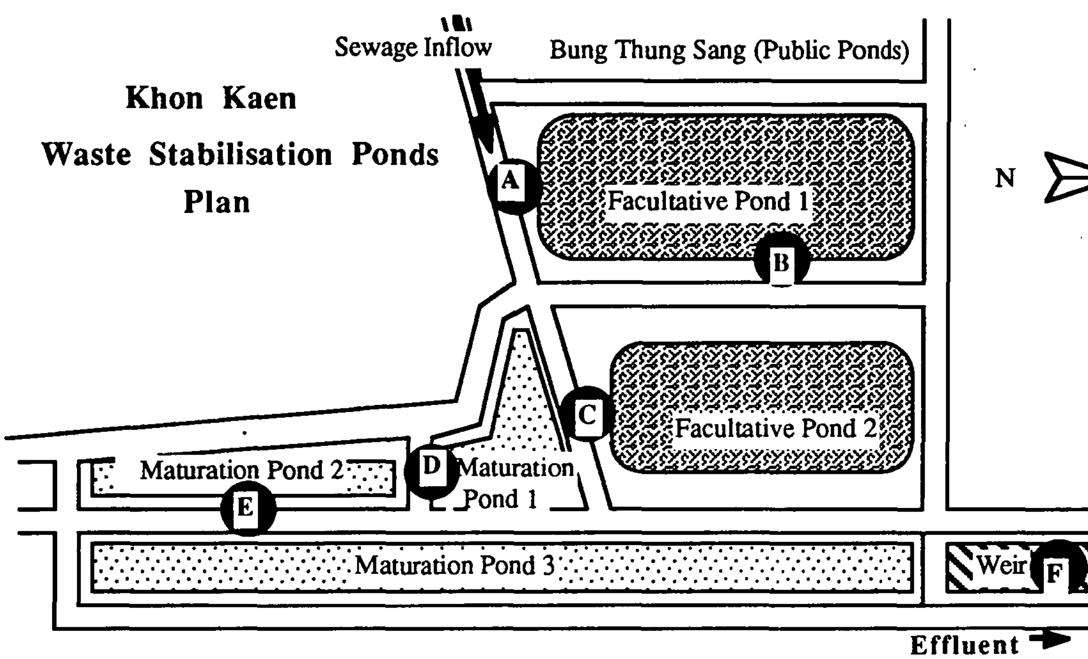
5.6 Measurement of the sewage flow

Sewage flow is measured by calculating the pumping hours. At the time of this study, the pumps were controlled manually instead of by an automatic switch. The daily quantity of wastewater inflow is calculated by multiplying the capacity of the pumps with the number of times of operation a day. This will be shown in cubic metres per day of wastewater flowing into the WSPs.

5.7 Wastewater sampling

The over-all sampling involved in this examination was carried out within a series of 12 weeks collecting wastewater samples from five ponds. The first sampling of wastewater was from the raw sewage influent (A). The second (B) and third (C) were from the effluent discharged from the first and second facultative ponds respectively, and the fourth (D), fifth (E) and lastly, the sixth (F) were from the final effluent of the three maturation ponds. The wastewater sampling sites in this study were selected in order to cover essential points in the plant, which will show pond performance in every stage the sewage passes through. The sampling sites for sample collection are shown in Figure 5.1.

Figure 5.1 The sampling sites in relation to the plant diagram.



5.8 Sampling procedure

Wastewater samples for the tests described in Table 5.1 were collected by grab sampling from the six defined station sites as shown in Figure 5.1. Wastewater samples for BOD_5, COD, DO, TSS, TDS, PO_4-P, NH_3-N, NO_3-N, and NO_2-N tests were collected from the sampling sites by using sampling bottles. Collection and testing of wastewater samples in this study was made on a bi-weekly basis starting from the 24th January 1993 and ending on the 26th April 1993.

The number of samples for each BOD_5, COD, DO, TSS, TDS, PO_4-P, NH_3-N, NO_3-N, and NO_2-N test totalled 432 samples. This was derived from the product of 24 collections multiplied by 3 wastewater samples and 6 collection sites.

The six sites investigated (A to F) are shown in Figure 5.1. The values of the nine parameters above were tested and three samples for each test were obtained to attain more accurate results.

The collected samples were examined immediately at The Environmental Health Centre Laboratory, Khon Kaen, Thailand. For pH and temperature, the tests were performed directly in the field during the period of collecting wastewater samples. One reading for pH and temperature was made at each site.

Tests for total coliform and faecal coliform were investigated at the same time as the other parameters. Due to the high cost involved, only one sample was collected from the three sampling sites instead of three samples as for the above nine parameters.

The equipment used for sampling BOD_5, COD, DO, TSS, TDS, PO_4-P, NH_3-N, NO_3-N, and NO_2-N was a wastewater sampler which was filled and separated into three sub-samples. These three samples were independently analysed and the result was used for calculating the average value for each determinant for that site on a given day.

The bottles used for BOD_5 and DO were the BOD_5 standard bottles. For the bacteriological tests of faecal coliform and total coliform, the standard sterilised glass bottles for bacteriological tests were used. Polyethylene bottles with a capacity of 2 litres were the containers used for the biota species tests.

Each wastewater sample, after being collected was immediately stored in a dark, iced container. After the last sample was collected, all samples in this iced container were simultaneously transferred to the laboratory room of the Environmental Health Centre which is about 2 km away from the sewage plant. The time taken to collect samples from all six sites was about 1.5 hours. The procedure for collecting samples from the selected sites followed the standard method from the APHA (APHA 1989).

For measuring temperature and pH, a thermometer and pH Meter were used. Other parameters were examined in the laboratory from the sewage samples received from the sewage plant. All of the parameters were investigated within a day of the completion of sample collection. This excluded the BOD_5 test since this has to be stored in an incubator at 20 ^{0}C for 5 days according to the standard method of BOD_5 testing procedure.

Sample collection started at 10 am on each testing day. The scheduled dates of investigation were every Monday from the 24th January, ending on Monday, 26th April 1993. This constituted a total of 24 time-tests during the summer months.

Equipment for BOD_5, DO, NH_3-N, temperature, pH, total and faecal coliform tests was supplied by the Environmental Health Centre Laboratory.

Measurements of NO_2-N, NO_3-N, PO_4-P, required a test kit (Palintest-photometer) from the Centre for Environmental Studies, University of Tasmania, Australia. This test kit has been calibrated and has provided a 95% statistically confident figure when compared to the results from conventional wastewater examinations (as illustrated in standard method APHA 1989).

The COD test used equipment from the Regional Water Supply Laboratory, Khon Kaen, Thailand. It should be noted that this examination in summer time was viewed to be significant since the Department Head of Environmental Health of the Khon Kaen City Council, who is in charge of this sewage treatment plant, requested a study of plant efficiency during the dry season.

During past plant observation, the Head of Environmental Health of the Khon Kaen City Council found that many problems emerged during summer (February to April), every year since the plant started operation in 1989 and more extensively in 1992. The problems of odour and algal floating mats in the sewage treatment ponds are the most common.

Because of this, the Khon Kaen City Council is also concerned about the efficiency of this sewage treatment plant which might be unable to purify sewage in compliance with the national effluent standard (KKM 1992b). One serious difficulty the city council has encountered was public complaints about the offensive odour from the sewage plant which, in addition, has been a breeding place for mosquitoes during hot season. The Khon Kaen City Council would like to know about the WSP system's performance and to understand how it works in order to find out a solution. Since this WSP plant is the biggest WSP sewage plant in Thailand, at present there is no document on this system or about any city WSP system in Thailand.

The Khon Kaen City Council in fact, requires sophisticated tests as proposed in this study. This should include a theoretical review of the WSP system. These challenging tasks seem to be beyond the capacity of the council and other local institutions in rural Thailand (Dr Thaveesuk Panpeng pers. comm.1993). With reference to Hess (1983), wastewater examination in any study in a developing country has a problem regarding availability of equipment supply and skilled persons. During the preparation phase of this study, these two major issues were addressed. Thus, the results of this study should provide useful results for the development of future laboratory tests required for WSP systems in Thailand.

5.9 Daytime parameter variation tests for WSPs

The second group of tests in this study attempt to ascertain a correlation between DO (mg/l), pH, ambient and water temperature (^{0}C) and light intensity (lux) in the five ponds, as these parameters vary in time. PO_4-P, NO_2-N, NO_3-N, NH_3-N and H_2S tests are also included when determining daytime variation . All of these tests are illustrated in Table 5.2 below. However, since there is a problem of limited equipment for testing PO_4-P, NO_2-N, NO_3-N, NH_3-N and H_2S, these parameters were tested only in three time slots at 6 am, 12 noon and 6 pm.

Table 5.2 Parameters involved in testing daytime variation.

Parameters	Times	Sites
Ambient temperature (°C)	6 am to 6 pm	five ponds (2 FPs and 3 MPs)
Water temperature (°C)	6 am to 6 pm	five ponds (2 FPs and 3 MPs)
Light intensity (lux)	6 am to 6 pm	five ponds (2 FPs and 3 MPs)
pH	6 am to 6 pm	five ponds (2 FPs and 3 MPs)
DO (mg/L)	6 am to 6 pm	five ponds (2 FPs and 3 MPs)
PO_4-P (mg/L)	6 am, 12 noon and 6 pm	five ponds (2 FPs and 3 MPs)
NO_2-N (mg/L)	6 am, 12 noon and 6 pm	five ponds (2 FPs and 3 MPs)
NO_3-N (mg/L)	6 am, 12 noon and 6 pm	five ponds (2 FPs and 3 MPs)
NH_3-N(mg/L)	6 am, 12 noon and 6 pm	five ponds (2 FPs and 3 MPs)
H_2S (mg/L)	6 am, 12 noon and 6 pm	five ponds (2 FPs and 3 MPs)

The sampling sites for all of the examinations in Table 5.2 were located at the centre of the five ponds. The test date was planned to be the 30th of March, 1993. The testing times for these parameters except PO_4, NO_3, NO_2, H_2S and NH_3 tests were designed to be every hour, collecting times starting at 6 am and ending at 6 pm and this constituted 13 hours a day when sunlight was still prevalent.

Collecting bottles used were similar to those used in the tests listed in Table 5.1. Methods of collecting and preserving samples followed the standard method (APHA 1989).

To obtain more precise results for the parameters investigated, each represented value of DO, NO_3, NO_2, NH_3, PO_4, and H_2S was taken from the average calculated value of the three samples collected. The DO values from the five ponds were collected 13 times (6 am-6 pm). Each of the DO values came from a computation of the three samples investigated. Since there were 5 ponds, 13 collections and 3 samples in each, then the total number of DO samples was 195.

The total number of samples in each of the tests NO_3, NO_2, NH_3, PO_4, and H_2S is 45, comprised three samples collected separately from five ponds, within the three designated times (6 am, 12 am and 6 pm). This figure of 45 for each parameter will be reduced to 15 when considering the average value of each determinant at a specific time on each pond.

The equipment used for NO_3, NO_2, NH_3, PO_4, and H_2S measurement was the Palintest kit which after calibration provided similar results as the standard method, APHA (1989), at 95 percent precision. All of the tests for NO_3, NO_2, NH_3, PO_4, and H_2S were conducted in the field immediately after samples were collected.

For DO tests, the method in Table 5.1 was used. The tests required assistance in field work, and also these tests needed transport for transferring samples to the laboratory. Such assistance was accomplished by coordinating with The Khon Kaen City Council, The Environmental Health Laboratory Centre, and Khon Kaen University which provided both personnel and equipment.

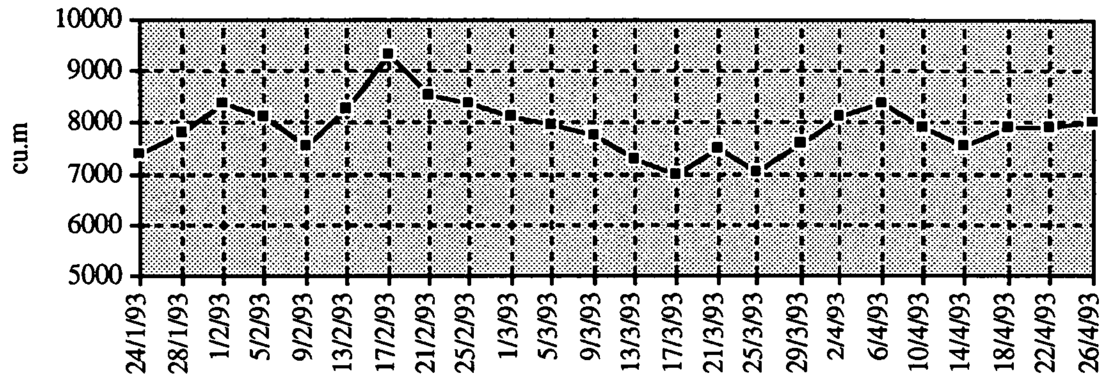
CHAPTER 6

RESULTS

6.1 Raw sewage flow

During the period of study, daily sewage flowing into Khon Kaen WSP ranged from 7,039-9,340 cu.m., as shown in Figure 6.1, with an average flow of 7914.3 cu.m. per day. This average flow figure constituted a sewage production per day of 72.4 litres per person or 376.7 litres per household (serving 109,241 population, 21,007 households).

Figure 6.1 Daily raw sewage incoming to Khon Kaen WSP.



6.2 Hydraulic retention time (HRT)

Raw sewage retention in five ponds averaged 33.5, 11.4, 2.5, 0.9 and 1.8 days in ponds 1, 2, 3, 4, and 5 consecutively. Total HRT during the study period averaged 50 days. The different HRT in the five ponds was due to the area of each pond, the first pond having the largest, and the fourth pond comprising the smallest area.

6.3 Temperature

Ambient temperature ranged from 25.1-32.5 ^{0}C, with an average of 29.2 ^{0}C. The temperature of the raw sewage was 20.0-27.^{0}C, averaging 25.8 ^{0}C, which was 3.4 ^{0}C lower than the average air temperature (Figure 6.2). The temperature of the five ponds was approximately the same, the average being 26.5, 26.6, 26.4, 26.4 and 26.4 ^{0}C from the first to the last pond (Figures 6.2 and 6.3). The temperature of all ponds was found to be a little higher than the raw sewage, but generally 2.7 ^{0}C lower than the ambient temperature. The minimum-maximum range of temperatures during the study period in the ponds were 24.0-29.2, 24.2-31.0, 20.0-31.8, 20.0-32.0 and 21.4-32.5 ^{0}C from pond 1 to pond 5.

Figure 6.2 Ambient, raw sewage and ponds 1 and 2 temperature.

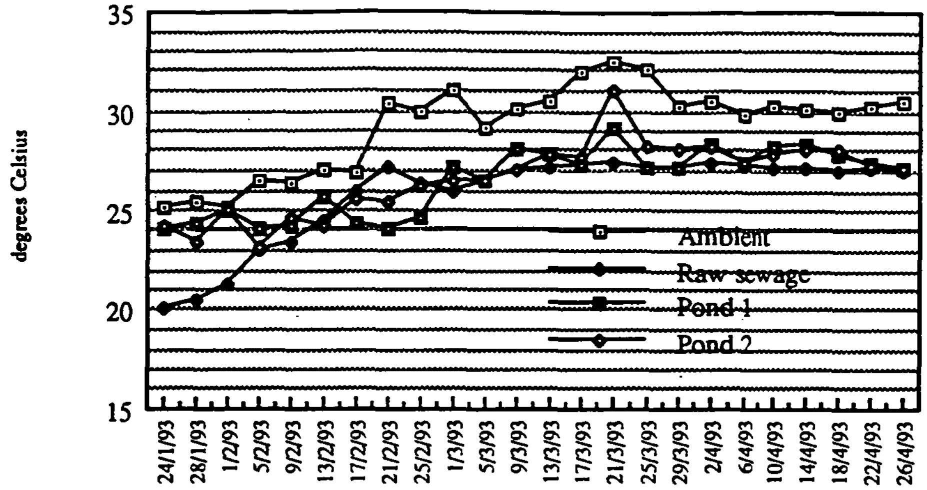
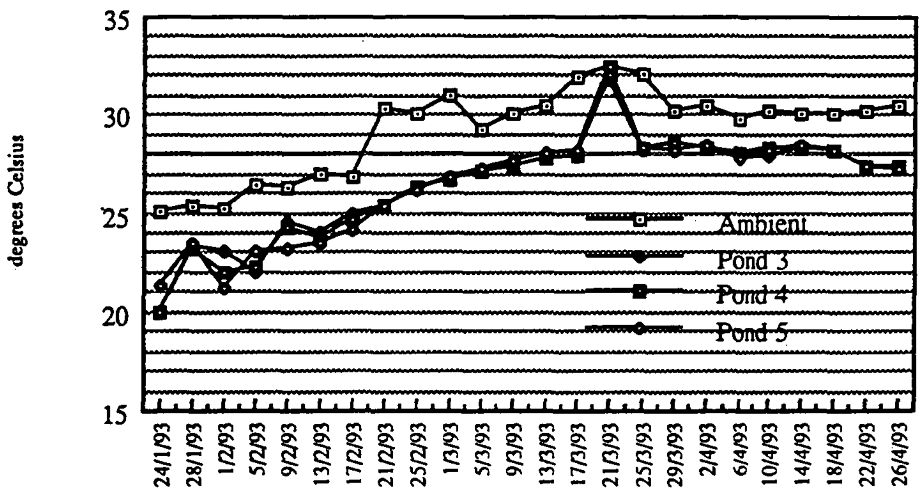


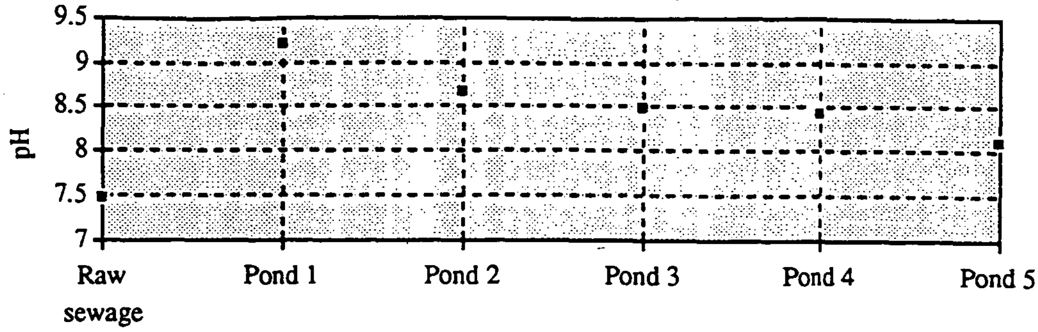
Figure 6.3 Ambient and ponds 3, 4 and 5 temperature.



6.4 pH

pH in the raw sewage was found to average 7.5. The first pond had the highest pH value averaging 9.2, while pH averaged 8.7, 8.5, 8.4 and 8.1 from the second to the last pond (Figure 6.4). The higher pH level in pond 1 mainly resulted from a larger content of algal cells. In the first pond algae using free dissolved CO_2 when dissolved CO_2 was exhausted, then carbonate and bicarbonate were used, therefore alkalinity reached a higher level. With an averaged pH of 8.1 in the final effluent from the plant, this pH level was meeting the country's standard (pH range 5-9).

Figure 6.4 pH levels in raw sewage and within the five ponds.



6.5 Total solids (TS), total dissolved solids (TDS) and total suspended solids (TSS)

TS level averaged 408.5 mg/L in raw sewage. The first pond had the highest value of TS, averaging 639.3 mg/L. From the second to the last pond, TS levels were different from each other, and slightly reduced, averaging 514.1, 488.2, 470.3 and 452.5 mg/L respectively (n 120, df 4/115, F 229.9, $p < 0.05$)(Figure 6.5). However TS concentration in the final effluent was higher than TS in raw sewage inflow. This was mainly due to the effect of heavy algal content in the ponds and the ponds being unable to remove TS effectively.

TS in the first pond was found loading at the lowest level, 3,237.2 kg/d, compared to the other ponds. The highest TS loading was in the second pond at 5,058.4 kg/d, whereas TS loading from the third to the last pond were at slightly lower in levels compared to the first pond which were 4,074.7, 3,867.5 and 3,725.5 kg/d consecutively (Figure 6.6). TS loading per unit area per day was at the highest level in the fourth pond, resulting from having the smallest area, averaging 10,743.0 kg/ha d. Whereas the first pond, occupying the largest area, loaded the minimal of 245.2 kg/ha d.

TDS level in raw sewage averaged 374.6 mg/L. The first pond had the highest value of TDS level, 519.3 mg/L, while ponds 2 to 5 had slightly different TDS levels, which were 422.3, 414.4, 407.4 and 401.0 mg/L respectively (n 120, df 4/115, F 145.4, $p < 0.05$)(Figure 6.5). The final effluent of TDS (401.0 mg/L) could meet the country's standard, the value of which was limited to not exceeding 500 mg/L (Table 2.5). Similar to TS, TDS levels in the final effluent was higher than in raw sewage, this was largely as a result of the inorganic salts generated by chemical reactions inside the ponds.

The first pond had the lowest value of TDS loading which was 2,968.4 kg/d. The highest TDS loading was in the second pond, 4,108.0 kg/d, and TDS loading ranged from 3,346.4, 3,283.0 and 3,019.0 kg/d from the third to the last pond respectively (Figure 6.6). With reference to these loading values, it appears that pond 2 had the highest inorganic salts.

TSS levels were lowest for raw sewage, averaging 33.9 mg/L. Average TSS levels varied from the first to the last pond, and decreased substantially, averaging 120.0, 91.8, 73.8, 62.9 and 51.5 mg/L (n 120, df 4/115, F 246.8, $p < 0.05$)(Figure 6.5). The final TSS (averaging 51.5 mg/L), outflowing from the last pond, would not meet the national effluent standard (the country's standard of TSS was not exceeding 30 mg/L, Table 2.5). This was similar to Hill and Shindala (1990) and McKinney's (1990) findings that TSS of most WSPs could not.comply with the effluent standard.

The second pond received the highest TSS loading, averaging 950.4 kg/d and the lowest level was in the first pond, 268.7 kg/d, while the third to the last pond averaged 728.3, 584.4 and 499.8 kg/d respectively (Figure 6.6). For TSS loading per unit area per day, the second pond had the highest level of 72.0 kg/ha d whereas the first pond had the lowest of 20.4 kg/ha d. From the third to the last pond, TSS loading was 55.2, 44.3 and 37.9 kg/ha d consecutively. Since TSS appears to represent the algal cells, the loading values of TSS can contribute the general view of algal density in each pond.

A common problem found in WSPs investigated by many authors (such as in Arthur 1983, Hess 1983, Mara 1988), was that solid contents were higher in effluent than in raw sewage (TS, TDS and TSS). Khon Kaen WSP also revealed the same problem. WSPs are largely dependent upon the biological ecosystem. Organic matter entering the pond and prevailing light and temperature cause the algal population, which exists symbiotically with bacteria, to increase considerably, therefore contributing in a major way to excess solids in the effluent.

Figure 6.5 TS, TSS and TDS levels in raw sewage and in the five ponds.

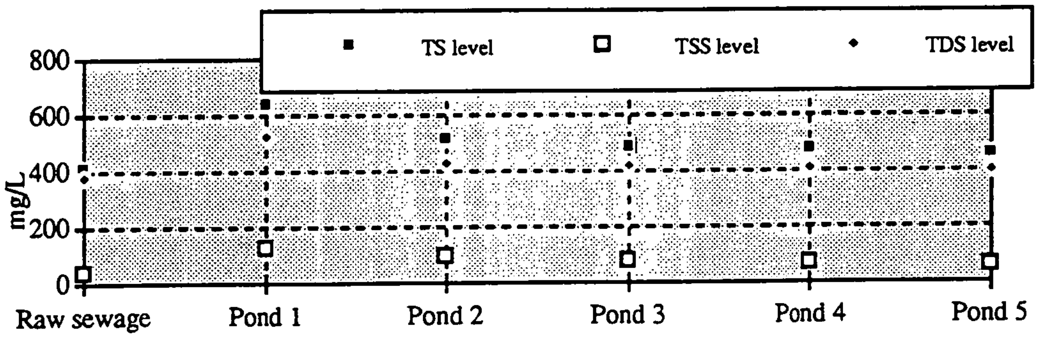
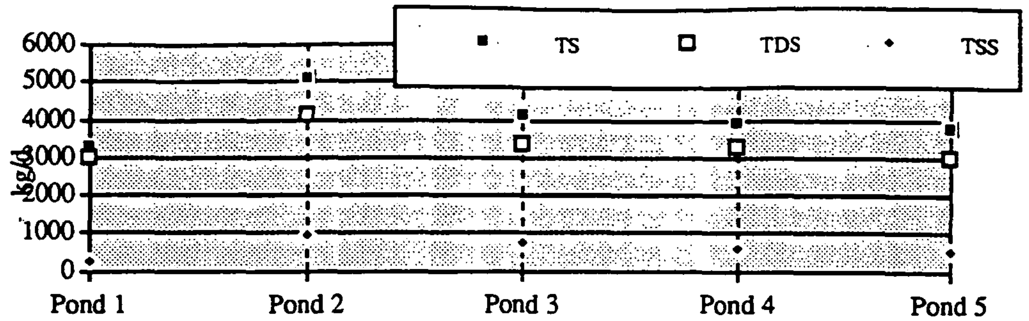


Figure 6.6 TS, TDS and TSS loading in the five ponds.



6.6 Biochemical oxygen demand (BOD) and chemical oxygen demand (COD)

BOD level in raw sewage ranged from 43.7-103.4 mg/L, averaging 74.2 mg/L. Average BOD content from the first to the last pond varied at 34.1, 23.3, 19.4, 17.6 and 14.8 mg/L respectively (n 120, df 4/115, F 164, p <0.05)(Figure 6.7). The final effluent BOD value from the last pond, averaging 14.8 mg/L, could meet the country's standard (not exceeding 20 mg/L, Table 2.5). From the raw sewage to final effluent, BOD levels were reduced significantly, 74.2 mg/L inflowed and 14.8 mg/L outflowed (n 23, t 18.1, p <0.05). However, one of the BOD values obtained on 17th February 1993 did not meet the country's standard (21.7 mg/L, see also Appendix 1).

Values for loading of BOD (organic loading) from the first to the last pond were 584.8, 270.1, 185.6, 155.1 and 140.2 kg/d respectively. Pond 1 had the highest load whereas pond 5 had the lowest (Figure 6.8). Areal BOD loading values were 44.3, 60, 187.4, 430.8 and 194.7 kg/ha d from pond 1 to pond 5. This different areal BOD loading in each pond was dependent upon pond area. Pond 4 had the lowest area, consequently receiving the highest areal loading.

COD levels in raw sewage ranged from 102-179 mg/L. Pond 1 had the highest COD level, averaging 229.5 mg/L, while ponds 2 to 5 had lower average COD levels which were 172.6, 131.8, 114.5 and 83.0 mg/L respectively (Figure 6.7). Average COD values in the final effluent were lower than inflowing COD (n 24, df 23, t 10.1, P <0.05). Average COD levels in the effluent were still high (83.0 mg/L), mainly resulting from a high algae content, as well as the processes causing high levels of TS and TSS in the effluent.

Figure 6.7 BOD and COD levels in raw sewage and in the five ponds.

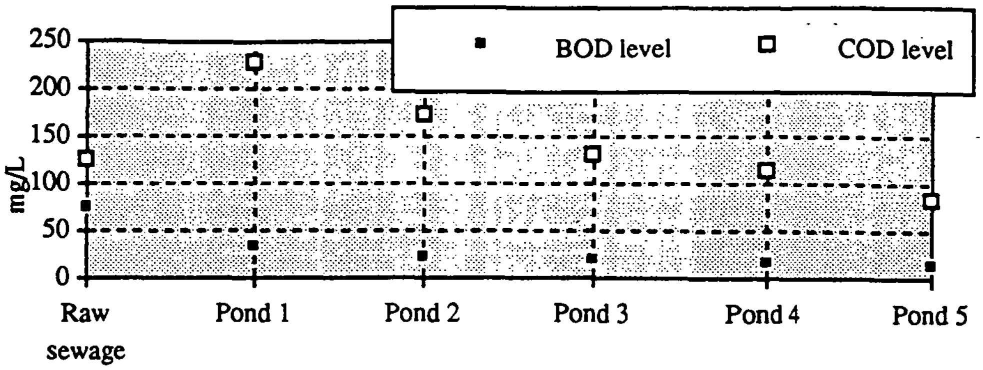
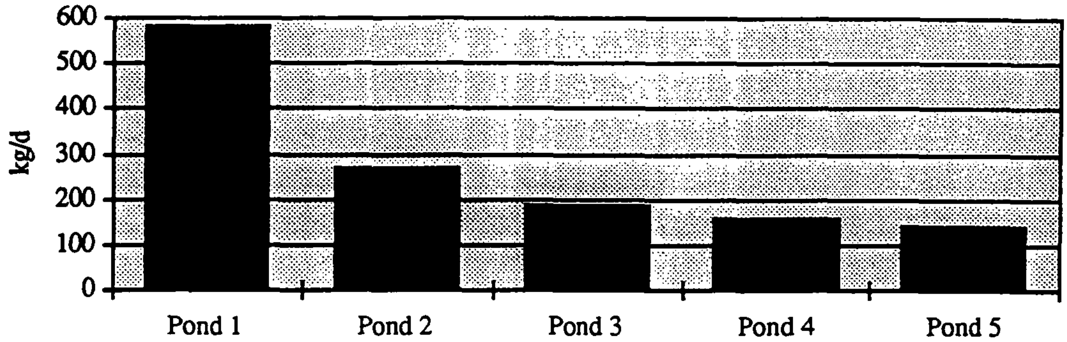


Figure 6.8 BOD loading in the five ponds.



6.7 Nitrogen and phosphorus levels

NH_3-N in raw sewage was high, averaging of 17.1 mg/L. For ponds 1 to 5 NH_3-N levels were 2.0, 0.9, 0.7, 0.4 and 0.5 mg/L consecutively (Figure 6.9). NH_3-N levels dropped significantly from high levels in raw sewage inflow between the first pond and the last pond (n 120, df 4/115, F 29, $p < 0.05$). Values for loading of NH_3-N in ponds 1 to 5 were 134.9, 15.5, 6.7, 5.3 and 3.3 kg/d. NH_3-N levels decreased intensively from 17.1 mg/L in raw sewage to 2.0 mg/L in the second pond, mainly due to (1) an uptaking of NH_3-N by algae and finally it was incorporated into new algal cells and (2) ammonia desorption to atmosphere.

NO_3-N levels in raw sewage averaged 0.2 mg/L. Ponds 1 to 5 showed no significant difference in NO_3-N levels, which were 0.2, 0.2, 0.3, 0.2 and 0.2 mg/L respectively (n 120, df 4/115, F 0.4, $P > 0.05$)(Figure 6.10). Referring to these values, it is obvious that the Khon Kaen WSP could not remove NO_3-N, when comparing inflow and outflow levels of NO_3-N (0.2 mg/L and 0.2 mg/L).

The constant NO_3-N levels remaining in the five ponds, could reflect a direction of a nitrification reaction oxidising NH_3-N->NO_2-N->NO_3-N. NO_3-N loading in the ponds averaged 1.7, 2.0, 1.8, 2.0 and 1.7 kg/d from the first to the last pond. These quite stable and low levels of NO_3-N in each pond also revealed that the nitrification reaction occurs to only a small extent. Most of NH_3-N was uptaken by algal cells rather than transforming into NO_3-N by the nitrification reaction.

NO_2-N in raw sewage averaged 0.04 mg/L. Ponds 1 to 5 had different values of NO_2-N levels (n 120, df 4/115, F 5.2, $p < 0.05$), which averaged 0.36, 0.60, 0.54, 0.38 and 0.30 mg/L respectively (Figure 6.10). NO_2-N in the effluent was higher than in the influent (n 24, df 23, t 6.1, $p < 0.05$). NO_2-N loading in ponds 1 to 5 was 0.3, 2.9, 4.7, 4.2 and 3.0 kg/d consecutively. NO_2-N levels in the ponds exhibited low levels, this also confirmed that nitrification occurred to a lesser degree in each pond since NO_2-N was an intermediate element of transforming NH_3-N to NO_3-N.

Figure 6.9 NH_3-N levels in raw sewage and in the five ponds.

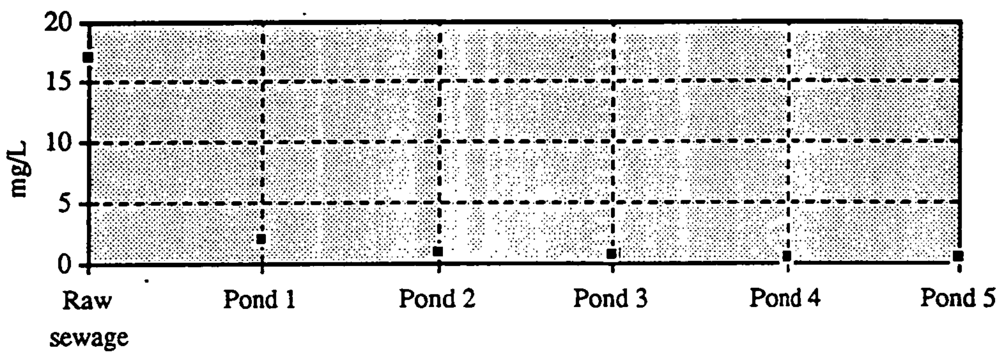
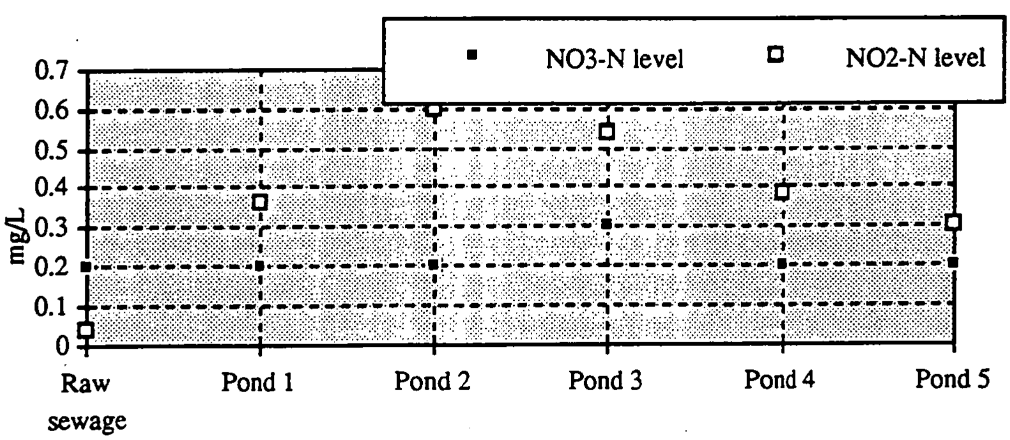


Figure 6.10 NO_3-N and NO_2-N levels in raw sewage and in the five ponds.

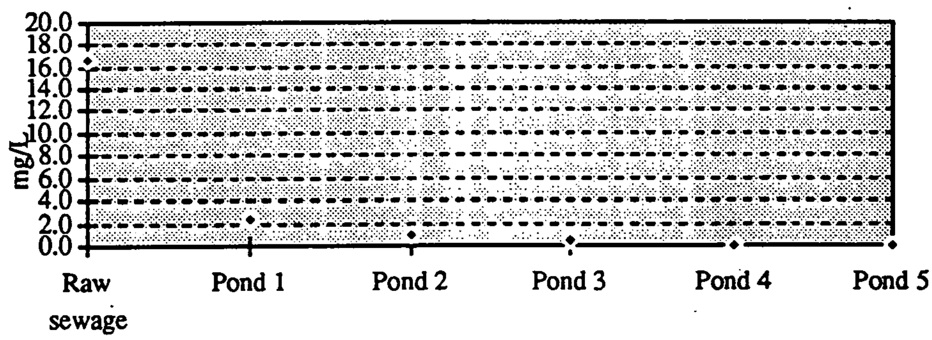


PO_4-P level was high in raw sewage, averaging 16.5 mg/L. There were different PO_4-P levels in the five ponds (n 120, df 4/115, F 445.4, $p < 0.05$).

From ponds 1 to 5, PO4-P levels averaged 2.5, 0.9, 0.4, 0.1 and 0.0 mg/L respectively (Figure 6.11). As PO4-P level was reduced from 16.5 mg/L in raw sewage to 0.0 mg/L in the final effluent, it is obvious that Khon Kaen WSP could reduce PO4-P efficiently.

The substantial reduction of this PO4-P level could reflect a biological reaction occurring which involves PO4-P being largely absorbed by algae. Pond 1 received the highest load of PO4-P, 130.1 kg/d, while from the second to the last pond the figures were 20.0, 7.2, 3.4 and 0.6 kg/d.

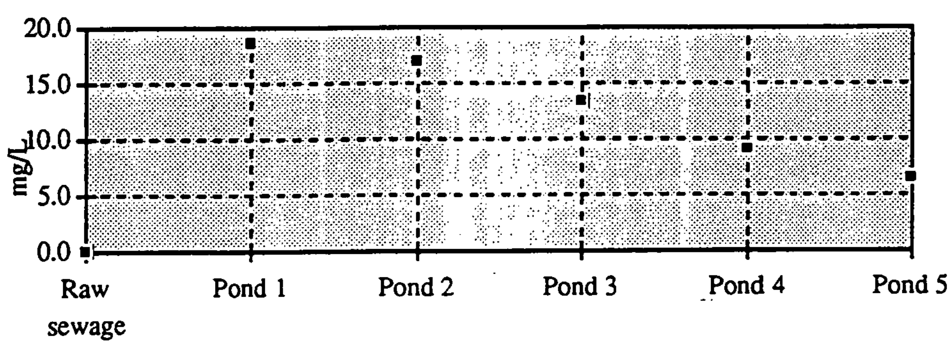
Figure 6.11 PO4-P levels in raw sewage and in five ponds.



6.8 Dissolved oxygen (DO) levels

DO in raw sewage was 0.0 mg/L. The highest level of DO was in pond 1 which averaged 18.5 mg/L, while in ponds 1 to 5 DO values were 18.5, 16.8, 13.3, 9.1 and 6.5 mg/L respectively (Figure 6.12). Pond 5 had the lowest content of DO, compared to ponds 1 to 4 (n 120, df 4/115, F 361, $P < 0.05$). These different levels of DO followed the TSS levels in each pond. Thus, the highest level of DO in pond 1 and the lowest level in pond 2, was dependent upon algal content, producing photosynthetic oxygenation. Pond 1 had the maximum algal content, while pond 5 had the minimum content.

Figure 6.12 DO levels in raw sewage and in the five ponds.



6.9 Total coliform (TC) and faecal coliform (FC) counts

Raw sewage had a value of TC averaging $3.5*10^7$ MPN/100 ml. TC in the second pond effluent (FP) averaged 1,097.2 MPN/100 ml and TC in the final effluent from the last pond (MP) averaged 120.7 MPN/100 ml.

An average count of FC in raw sewage was $1.3*10^7$ MPN/100 ml. FC values from the second pond (FP) and the last pond (MP) were 483.1 and 48.3 MPN/100 ml respectively.

Referring to the country's standard which limited TC numbers to not exceeding 5,000 MPN/100 ml, the Khon Kaen WSP could reduce TC numbers to the effluent standard. This also happened with FC. The effluent standard for FC numbers (not exceeding 1,000 MPN/100 ml) in the effluent, could be met. Further, there was no sample in this study having TC and FC counts over the standard, thus it appears that this plant has a good performance in TC and FC removal.

6.10 Efficiency of Khon Kaen WSP

BOD and COD removal

Khon Kaen WSP plant was totally capable of BOD reduction, from ponds 1 to 5, with an average of 80.1% (74.2 mg/L initial influent and final 14.8 mg/L effluent). Pond 1 had the highest capability of BOD reduction, achieving 54.0%, while ponds 2, 3, 4 and 5 achieved 31.7%, 16.7%, 9.3% and 15.9% BOD reduction respectively (Figure 6.13). Pond 1 and pond 2, facultative ponds (FPs), together reduced the BOD content by 68.6%. Ponds 3, 4 and 5, maturation ponds (MPs), as a whole, were capable of BOD reduction by 36.5% (Appendix 1).

With the Khon Kaen plant having the capacity of an average BOD reduction of 80.1%, this value fell into the range mentioned in Chapter 3 (Table 3.3) which was mainly 70-90% BOD removal. Compared to a WSP at a similar altitude in Malaysia (Bradley 1983) which had a capacity of 79% BOD reduction, the Khon Kaen WSP's capacity is very similar.

However, ponds 1 and 2, FPs, which were purposely designed for reducing BOD content, seemed to perform less efficiently in BOD removal than would be expected from documents discussed in the literature review in Chapter 3. The BOD level in the first FP averaged 34.1 mg/L and in the second FP, averaged 23.3 mg/L. The BOD level in the effluent from these FPs, was slightly over the standard (20 mg/L).

For the FPs, Yanez equation (1980) gave the expected value of effluent BOD from the FPs as 29.7 mg/L (Table 4.9). This model's value is closer to the actual BOD value (23.3 mg/L) than any other model. Other models, such as Mara (1976) predicted 4.1 mg/L; Marais and Shaw model A (1961) 71.7 mg/L and Marais and Shaw model B (1961) 7.9 mg/L (Table 4.10).

The conclusion about the equations predicting BOD_5 effluent levels was in general advocated to Yanez model which gave the closest value to the actual BOD_5 level.

However, there are some points worth noting about such equations only representing the guide values of effluent BOD_5. Values were not necessarily the same as the actual values but they were the expected values after constructing the plant. Sewage characteristics, algal species and other environmental factors also influenced the effluent BOD_5 levels. So, the equations were only effective in particular regions not universally. Also, this study was only conducted in the summer months. In rainy and winter seasons the actual values of effluent BOD_5 would be different. Therefore, Yanez model was analysed and found suitable in this case only in the summer months.

All of the models mentioned in this study were worthy of further investigation, related to this plant, in every season throughout the year. Any attempt to innovate a new formula for this plant by recording historical results was found to be preferable by this study. Such an exercise will eventually lead to attaining a more precise BOD5 equation for this plant and might be applied to a similar location, elsewhere.

From percent BOD reduction above, it seemed that the pond's area was one of the major factors influencing BOD reduction. For instance Pond 4, the smallest pond, had the lowest percent in BOD removal compared to the other ponds. Referring to Hess (1983) BOD loading in any WSP should not exceed 400 kg/ha d but this pond received a loading as high as 430.8 kg/ha d, which might be beyond the effective capacity of this pond in BOD removal.

The models mentioned in Chapter 4 gave the permissible organic loading (BOD) for the Khon Kaen WSP, which would allow this plant to perform at maximum efficiency in removing BOD. Six models recommended BOD loading as summarised in Table 6.1, the values of BOD loading varied from 275.0-876.4 kg/ha d. At Khon Kaen WSP plant, ponds 1 to 5 were loaded at 44.3, 60.0, 187.4, 430.8 and 194.7 kg/ha d respectively and a capacity of BOD removal of 54.0%, 31.7%, 16.7%, 9.3% and 15.9% consecutively.

It was quite clear that with the lower loading of BOD, the ponds could perform effectively in removing BOD. The first pond, with less BOD loading, could reduce BOD better than those more highly loaded.

However, one could not conclude that the areal loading was the only factor determining the efficiency degree of BOD removal. This was not an absolute conclusion, it was one of the factors influencing the BOD removal rate. Since Khon Kaen had a pretreatment unit in each household, so the readily degraded wastes were treated and let the less readily wastes pass into the WSP plant, the BOD reduction efficiency in this plant then differed from some other WSP plants which received raw sewage without pretreatment.

For similar reasons, the first pond had the highest BOD reduction while the proceeding ponds had lower. This difference could depend on the easily biodegradable characteristic of organic wastes in each pond as mentioned above. So, such a description on the matter of BOD loading relating to BOD reduction efficiency was only a general view of what was happening inside the ponds of this plant. Another important point to note was that this study only attempted to ascertain some consistency in what some authors like Hess (1983) documented in terms of loading factors influencing the BOD removal efficiency.

Table 6.1 Recommended BOD loads and the actual loading of Khon Kaen WSP plant.

Models	BOD loading (kg/ha d) for any WSP	Actual BOD loading (kg/ha d)
1. McGarry and Pescod Model (1970)	876.4	pond 1= 44.3
2. Mara's model A (1976)	586.88	pond 2 = 60.0
3. Mara's model B (1976)	314.0	pond 3 = 187.4
4. Arceivala model (1970)	275.0	pond 4 = 430.8
5. The Asian Institute of Technology (1981)	626.0	pond 5 = 194.7
6. Arthur (1983)	374.0	-

BOD loading values given in Table 6.1 or from the empirical equations in Chapter 3 were mostly higher than the actual values. With the recommended BOD loading of these empirical equations being inappropriate to Khon Kaen WSP, other suggestions for BOD loading were taken from the research results of other authors.

It was found that BOD loading of Khon Kaen WSP (30.2 kg/ha d) was similar to BOD loading ranges recommended by some authors (Allum 1955, Metzler 1959, Pierce 1960, Clark and Kalda 1961, Middlebrooks *et al.* 1979) which ranged from 22-56 kg/ha d.

Typical BOD loading values for tropical countries were suggested by Hess (1983) at 150-400 kg/ha d; Gloyna (1971) at 150-315 kg/ha d; and Metcalf and Eddy (1972) at 16-56 kg/ha d. Only the latter range was appropriate to the actual loading in Khon Kaen WSP.

In summary, such empirical models as analysed in Chapter 4 cannot effectively be applied to Khon Kaen WSP. Only Yanez model for the first FP could closely reflect the actual situation at this plant. The recommendation from other investigators proposed BOD loading ranges that were similar to the Khon Kaen WSP current situation, is by Metcalf and Eddy (1972) and Middlebrooks *et al.* (1979).

Of the factors influencing WSP efficiency in BOD removal, organic loading (kg BOD/ha d) seemed to be a major factor at the Khon Kaen WSP. According to Figure 6.13 and Appendix 1, it was found that the percentage of BOD removal decreased whenever the organic content was increased.

Pond 1 had the lowest organic loading (44.3 kg BOD_5/ha d) and had the highest percentage of BOD reduction. On the other hand pond 4 had the highest BOD load (430.8 kg/ha d) and could remove BOD to the lowest percentage. This was in agreement with Barsom (1973), O' Brien (1978), Mara (1982) and Banerji and Ruess (1987) who documented that the organic loading to a pond was the major factor influencing a WSP's efficient BOD reduction.

Another factor associated with BOD removal efficiency was pond depth. Although Khon Kaen WSP ponds had the same depth (both FPs and MPs), it appeared that the ponds' efficiency, as compared with percent BOD removal, still varied as shown in Figure 6.13. This study, therefore, did not have enough data of varying pond depth to compare against percent BOD reduction. However, it seemed that 2 m, together with a HRT of 33.3 days, was reasonable for the first pond. These two factors enabled it to have the capacity to remove on average 54% of BOD. This relationship of percentage BOD removal and pond depth apparent in the Khon Kaen WSP also agreed to findings by Brick (1961), Fleming (1962), Mara (1976) and Widmer (1981).

To a large extent, however, the WSP performance in reducing BOD contents is dependent on many factors (as many authors have mentioned); physical features, e.g. appropriate area, depth and shape of the pond, and HRT; biological factors e.g. microorganism content, algal, bacterial species; and biochemical factors e.g. pH, NO_3-N and PO4-P levels. From an examination of the Khon Kaen plant, one can conclude that a combination of such factors as the pond's loading, depth and HRT all influence its efficiency in removing BOD. The depth of 2 m at this plant, with low BOD loading and a long HRT, seem to lead to increased BOD removal. To achieve increased efficiency in BOD removal in pond 4, for example, expanding the pond's area would increase HRT and lessen BOD loading.

Overall COD removal at the plant was on average 33.9% (125.5 mg/L initial influent and 83.0 mg/L final effluent). Pond 1 could not remove COD, but COD was increased by 82.9% (125.5 mg/L influent and 229.5 mg/L effluent), whereas ponds 2, 3, 4 and 5 were capable of 24.8%, 23.6%, 13.1% and 27.5 % COD removal (Figure 6.13). Ponds 1 and 2, collectively, were unable to reduce COD, which increased by 37.5%. Ponds 3, 4 and 5, wholly, accomplished a positive COD reduction of 51.9% (Appendix 1).

The COD level in pond 1 increased to nearly twice the COD level in the sewage influent. This was mainly due to increased solids in pond 1 raising the COD level. Ponds 2 and 3 removed COD continuously, but the final COD effluent was still high (83.0 mg/L).

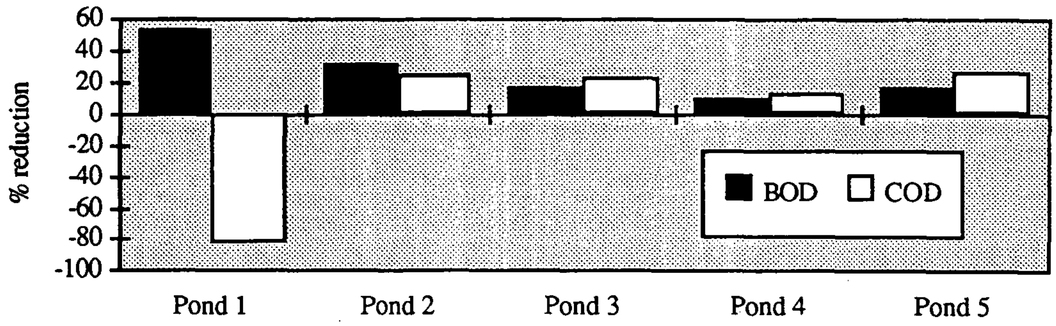
This presence of solids was mainly due to algal cells. Further, pond 1, having the highest percentage of BOD removal, in contrast, had the lowest efficiency in COD reduction (-82.9%). Of the five ponds, pond 5 achieved the highest percentage of COD removal (27.5%). Comparing COD loading from ponds 1 to 5, pond 2 had the highest load but it still accomplished a higher percentage of COD removal (24.8%) than pond 1 (-82.9%).

COD loading did not appear to be a factor influencing percent COD removal in each pond. Since COD levels in the five ponds seemed to correlate with TS and TSS levels (Appendix 1), one might assume the degree of COD reduction in each pond might be comparable. However, the TS and TSS values in pond 4 were lower than those of ponds 2 and 3, but pond 4 still had a lower capacity for COD reduction. So, TS and TSS levels were not the main factors influencing COD reduction. It was also not clear that pond area correlated with COD reduction. Since COD refers to all chemically oxidised matters, both living and non-living existing, within the ponds, the factors influencing the efficiency of COD reduction are difficult to determine.

Comparing COD and BOD reduction in this plant, the efficiency of COD removal was lower than that for BOD (COD 33.9% and BOD 80.1%). The lower percent of COD reduction was similar to findings by Novak (1976) and Silva, Mara and de Oliveira (1987). They anticipated that the percentage of COD reduction in WSPs would be 51.8% and 69.0% respectively, which was higher than recorded at this plant (33.9%).

However, compared to the Khon Kaen WSP, their WSPs had higher levels of COD in the effluent (averaging 107.3 and 109.0 mg/L, whereas Khon Kaen was 83.0 mg/L). This lower percentage COD reduction at Khon Kaen WSP fell into the range of 30-65% which was the acceptable efficiency level approved by The Environmental Protection Agency (USA).

Figure 6.13 Percentages of BOD and COD reduction.



Solid removals

Total solid (TS) was not reduced by the plant but increased by 10.8% (408.5 mg/L initial influent and 452.5 mg/L final effluent). In Pond 1, TS increased by 56.4%, whereas ponds 2 to 5 decreased TS by 19.6%, 5.0%, 3.7% and 3.8% respectively (Figure 6.14). Ponds 1 and 2, wholly, could not reduce TS, but increased it by 25.8%. Ponds 3 to 5 were collectively capable of a positive TS reduction of 11.9% (Appendix 1).

TDS was also increased at the plant by 7.0% (374.6 mg/L initial influent and 401.0 mg/L final effluent). In Pond 1, TDS increased by 38.6%, whereas ponds 2, 3, 4 and 5 decreased TDS by 18.7%, 1.9%, 1.7% and 1.6% (Figure 6.14). Pond 1 and pond 2, collectively, could not reduce TDS (-12.7%). Ponds 3, 4 and 5 were able to decrease TDS by 5.0% (Appendix 1).

Total suspended solid (TSS) was increased at the plant by 51.9% (33.9 mg/L initial influent and 51.5 mg/L final effluent). In Pond 1, TSS increased by 253.9%, whereas ponds 2, 3, 4 and 5 reduced TSS by 23.5%, 19.6%, 14.8% and 18.1% respectively (Figure 6.14). Ponds 1 and 2, collectively, could not decrease but increased TSS by 170.8%. Ponds 3 to 5, as a whole, were able to reduce TSS by 43.9% (Appendix 1).

The pattern of percentage TS and TSS reduction, was similar to COD removal. In Pond 1, TS and TSS were increased, leading to a higher value of COD. Since COD is a value reflecting chemical oxidisation, then, all solids are included together and combined with other substances to form COD values. In ponds 2 to 5 solid levels reduced, as these ponds appeared to positively remove solids and therefore COD reflected a positive percentage reduction.

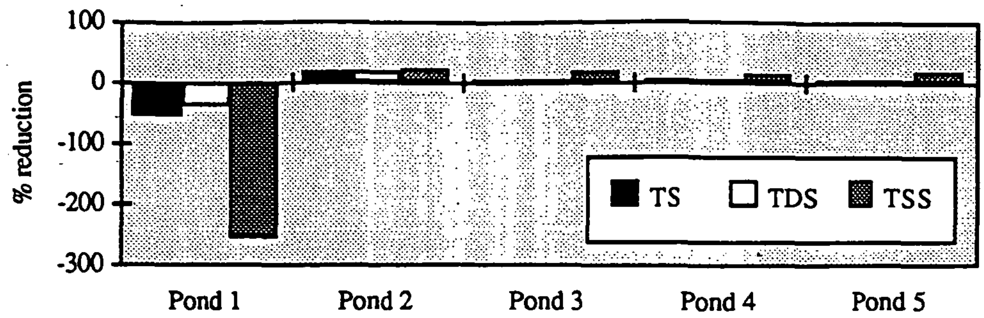
Pond 4 had the lowest percentage of solid reduction, similar to the case with BOD removal. According to loading per unit area, pond 4 received the highest loads of TS (10,743.0 kg/ha d) compared to the other ponds. It also occupied the smallest area which seemed to be a limiting factor to its efficiency in reducing solids. An observation from this study was that this pond appeared to have a high algal bloom content and an unpleasant smell could be detected. Clogging inside this pond was often noted by the plant officers. The offensive odour generated by this pond was likely to be from an accumulation of solids which resulted in an anaerobic condition.

Being unable to reduce solids, particularly TSS, was one of major problems of this plant. The country's standard limited the TSS value in the final effluent to not exceeding 30 mg/L, and it was found that only two samples during the study period had TSS values complying with the standard (29 mg/L on 1st February 1993 and 30 mg/L on 18th April 1993). 91.6% of TSS samples still had values over the standard (Appendix 1).

The main concern of this plant is its ability to reduce TSS levels in the WSPs. As shown in Figure 6.14, it was found that TSS was reduced substantially with each subsequent pond (from 1st pond TSS 120 mg/L till the 5th pond TSS 51.5 mg/L). Aiming at decreasing TSS to meet the standard, this WSP would need new ponds preceding the former WSPs or would require other measures e.g. initiating aquaculture for reducing TSS.

The inability to reduce TSS in Khon Kaen WSP was in contrast to other WSPs' performance from evidence in the USA, which could reduce TSS by 40-90% (e.g. Fall 1971, Oswald 1976, Novak 1976, Banerji and Ruess 1987), from Malaysia where TSS was decreased by 50%, and from Mexico where TSS was reduced by 87.5% (De la O and Martinez 1976). However, the TSS levels at Khon Kaen WSP were similar to those documented by Orgeron (1976) where a WSP increased TSS by 44.3%. The removal of TSS involves many factors: local environment such as temperature, sunlight, pond dimension, biota species, nutrients and so on. Insufficient data from the other plants mentioned makes it impossible to investigate the factors affecting different rates of TSS removal between Khon Kaen WSP and those plants.

Figure 6.14 Percentages of TS, TDS and TSS reduction in Khon Kaen WSP.



Nitrogen removal

NH_3-N could be effectively removed in parts of the plant with an average percentage of 97.1 (17.1 mg/L initial influent and 0.5 mg/L final effluent). Ponds 1, 2, 3, and 4 could reduce NH_3-N by 88.3%, 55.0%, 22.2% and 42.9% respectively, while pond 5 increased NH_3-N by 25.0% (Figure 6.15). Ponds 1 and 2, collectively, reduced NH_3-N by 94.7%, while ponds 3, 4 and 5, as a whole, decreased NH_3-N by 44.4% (Appendix 1).

NO_3-N was not removed by the plant, but was slightly increased by 7.8% in the final effluent (0.2117 mg/L initial influent and 0.2283 mg/L final effluent). In ponds 1, 3 and 5, NO_3-N was increased by 17.9%, 12.7%, and 9.36% respectively. In ponds 2 and 4, NO_3-N was decreased by 11.0% and 16.6% (Figure 6.15). Ponds 1 and 2, collectively, increased the NO_3-N level by 0.05%. Ponds 3, 4 and 5, together, decreased NO_3-N by 8.8%. (Appendix 1).

NO_2-N was not removed by the plant but was wholly increased in the plant by 650% (0.04 mg/L initial influent and 0.30 mg/L final effluent). In ponds 1 and 2, NO_2-N was increased by 800.0% and 66.7% respectively (pond 1, 0.04 mg/L influent and 0.36 mg/L effluent, pond 2, 0.36 mg/L influent and 0.60 mg/L effluent). Ponds 3, 4 and 5 could reduce NO_2-N by 10.0%, 29.6% and 21.1% respectively (Figure 6.15). Ponds 1 and 2, as a whole, increased NO_2-N by 1,400%. Ponds 3, 4 and 5, collectively, could reduce NO_2-N by 50.0% (Appendix 1).

Referring to the above three groups of nitrogen reduction, it could be assumed that NH_3-N was transformed to NO_2-N and NO_3-N. As Middlebrooks *et al.* (1979) noted, NO_2-N levels increased whenever HRT was expanded, and consequently NO_3-N was later increased as a result of NO_2 nitrification. With an HRT of 44.9 days in pond 1 and pond 2 this reaction appeared to occur. Also, percent decrease and reduction of NO_2-N and NO_3-N in the ponds mentioned above could imply the direction of nitrification reaction occurring in the ponds.

Thus NH_3-N was greatly reduced and NO_2-N greatly increased in the first two ponds. In pond 3, NO_3-N was detected at a higher level (0.3 mg/L) than in the other ponds. It is possible that this was due to (1) a greater degree of transforming NO_2-N to NO_3-N and/or (2) algal die-off releasing NH_3-N->NO_2-N->NO_3-N. It seems likely that the first case applied, since solids content in the third pond was at a lower level. This could be asserted by (1) according to observation made at pond 3 during the study, there was no obvious appearance of algal die-off to release cellular NH_3-N to NO_3-N and (2) ponds 1 and 4 had a considerable algal die-off but exhibited a lower level of NO_3-N (0.2 mg/L).

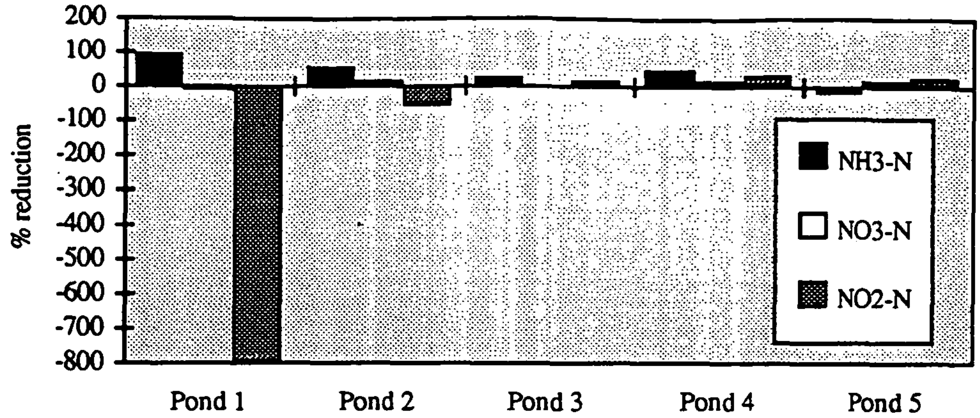
In general, nitrification was expected to occur in the first two ponds, but when comparing the major reduction of NH_3-N level between nitrification and algal uptaking, such nitrification reaction was likely to have a minor role in transforming NH_3-N to NO_3-N. The main sink of NH_3-N would be the result of algal culture uptaking NH_3-N, leaving only a small portion being transformed to NO_3-N. This could be evidenced by the lower levels of NO_2-N and NO_3-N in each pond.

Pond 5 was found to increase NH_3-N levels. This could have been a consequence of large numbers of fish inhabiting this pond, since fish also produce ammonia-nitrogen. Another influence on increased NH_3-N levels in this pond was solids settling on the pond bottom, as could be seen from TSS level was at low level in this pond, and therefore ammonification was expected to occur.

The NO_3-N levels remaining in each pond were due to the ponds having a high level of DO, consequently denitrification could not be expected to occur. Also, since there was NH_3-N available in the ponds, algal culture would preferably take it up as a nutrient source rather than from the NO_3-N source.

Although this WSP plant could not remove NO_3-N, the level still remained low at 0.2 mg/L. It was observed during the study that the receiving stream had no algal bloom occurring along the water body. This suggests that although this plant did not have the ability to reduce NO_3-N, the levels in the discharge did not cause river eutrophication.

Figure 6.15 Percentage of nitrogen removal in the five ponds.



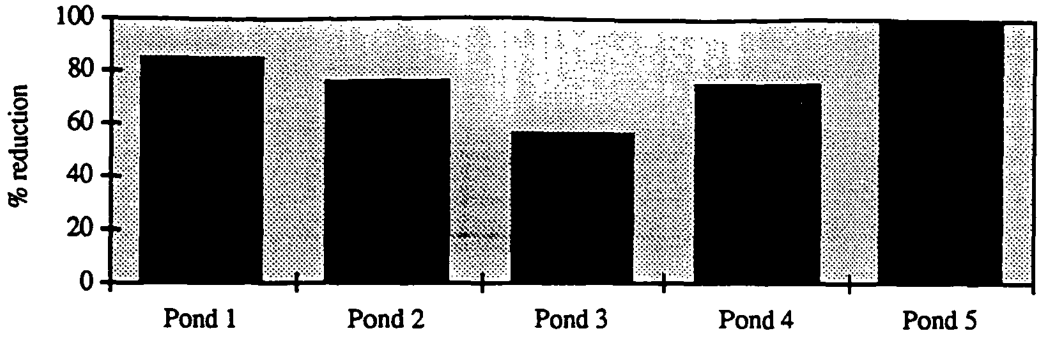
Phosphorus removal

The plant was able to remove phosphorus by 100% (16.5 mg/L initial influent and 0.0 mg/L final effluent). Ponds 1 to 5 could reduce PO_4-P at 84.8%, 76.0%, 55.6%, 75.0% and 100% respectively (Figure 6.16). Ponds 1 and 2 could reduce 94.5% and 100% consecutively (Appendix 1).

As one of the limiting nutrients to plant growth, phosphorus was likely to be utilised by algae culture. PO_4-P entering the plant, with a concentration of 16.5 mg/L, was reduced to 2.5 mg/L at the first pond, and removed entirely by the last pond. The level of PO_4-P in any watercourse would limit the number of aquatic plants. The zero discharge from Khon Kaen WSP would be of benefit to the control of eutrophication in the receiving water. Further, it can be concluded that Khon Kaen WSP is more efficient plant in reducing PO_4-P compared to other WSPs which had 90-95% (Assenzo and Reid 1966, Lyman 1970), 75% (Bucksteeg 1987) and 26% reduction (Oswald 1976).

However, the reduction of PO_4-P here, was measured in soluble PO_4-P so the statement of total reduction of PO_4-P was as soluble PO_4-P removal. When considering PO_4-P as total phosphorus there was still phosphorus leaving this WSP plant in the form of cellular compound of algal cells.

Figure 6.16 Percentage of phosphorus removal in the five ponds.



Bacteria removal

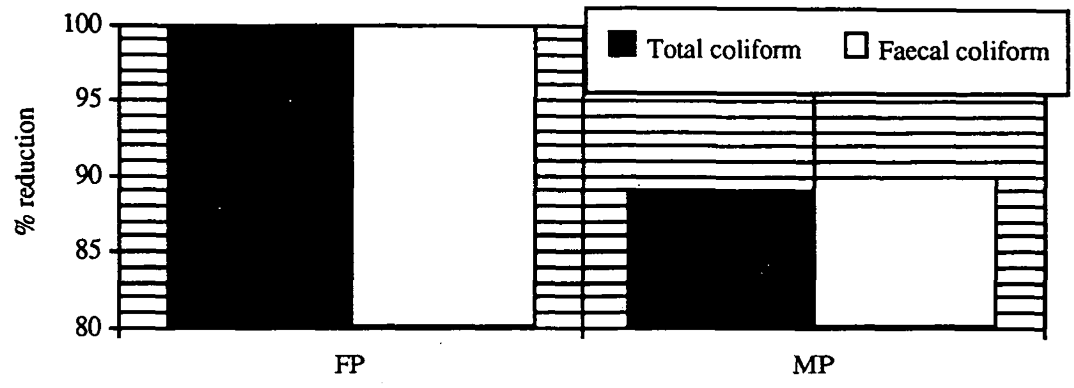
Total coliform (TC) in the plant was reduced by 99.9% ($3.52*10^7$ MPN/100 ml initial effluent and 120 MPN/100 ml final effluent). Ponds 1 and 2 as FPs, collectively, could reduce TC 99.9%. Ponds 3, 4 and 5, as MPs, could decrease TC by 89.0% (Figure 6.17).

Faecal coliform (FC) was reduced similarly up to 99.9% ($1.29*10^7$ MPN/100 ml initial effluent and 48.3 MPN/100 ml final effluent). Ponds 1 and 2, as a whole, could reduce FC by 99.9%, and ponds 3, 4 and 5, together, could reduce FC by 90.0% (Figure 6.17).

The Khon Kaen WSP plant operated effectively in bacterial removal, in a similar way to the studies documented in Table 3.4. Conventionally, the FP was designed to reduce mainly organic content in sewage but in this case the FP could reduce both TC and FC substantially. Given the country's standard of TC and FC values not exceeding 5,000 and 1,000 MPN/100 ml respectively, it could be seen that TC outflowing from the second FP (1,097.2 MPN/100 ml) could meet the standard. This also was the same for FC levels where FC on average outflowing from the second FP was only 483.1 mg/L.

Further it was found that none of the TC and FC samples discharged from the last FP were over the standard. In contrast, MPs which are aimed at reducing bacteria, displayed inferior performance in bacterial reduction compared to FPs, in that their capacity was only 89.0% for TC removal (1,097.2 MPN/100 ml influent and 120.7 MPN/100 ml effluent) and 90.0% for FC removal (483.1 MPN/100 ml influent and 48.3 MPN/100 ml effluent). However, the MPs seemed to have some function in BOD purification activities, in that when discharged from the FP the final BOD level met the standard (Appendix 1).

Figure 6.17 Percentage of total coliform and faecal coliform removal in FP and MP.



6.11 Loading and removal relationships

Using a simple linear correlation for examining the direction of relationships between major parameters, it was found that some parameters correlated to each other. Major parameters were analysed in terms of correlation on the basis of areal loading and removal relationship. Spearman's correlation analysis was used to identify any loading-removal relationship of the main parameters concerned. In this study the number of samples tested were the same 24 samples in all parameters, so that any correlation occurring was referred to the correlation coefficient n = 24, df = 22, at $r_{0.05\ (1),\ 22} = 0.344$.

6.11.1 *BOD loading and removal*

BOD loading and removal in pond 1 demonstrated a significant positive correlation, as shown in Figure 6.18 (r 0.931, $p < 0.05$). Also, ponds 2 and 3 exhibited positive correlation between BOD loading and removal (r 0.551, 0.577, $P < 0.05$), but no correlation was shown in ponds 4 and 5 (r 0.328, 0.277, $p > 0.05$).

Pond 1 showed almost a hundred percent positive correlation between the BOD applied and its removal. Ponds 2 and 3 had lesser degrees of correlation. Ponds 4 and 5 had no significant correlation between BOD loading and removal. Referring back to the failure of the BOD loading design equations, this actual relationship between BOD applied and removed was an alternative for use in WSP design.

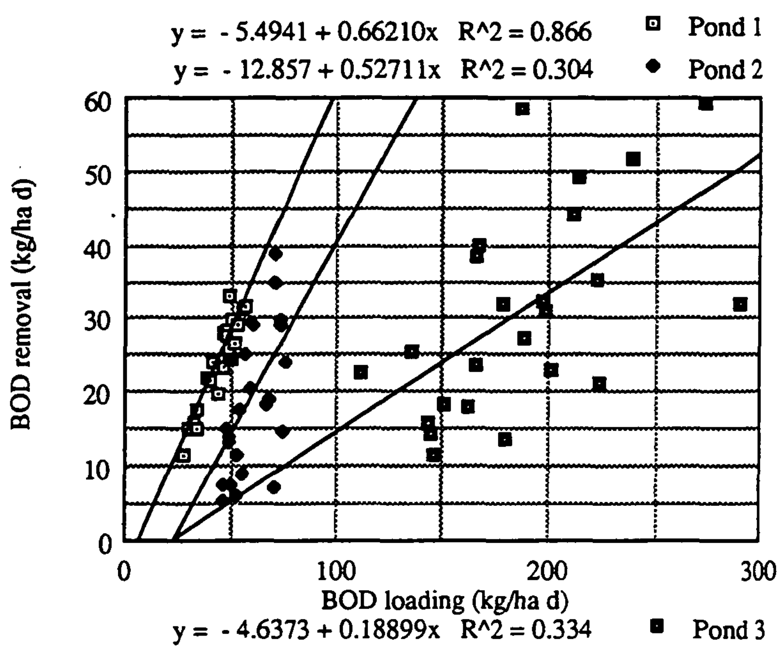
According to Middlebrooks (1987) and Jones (1993), a simple linear correlation equation would be the best alternative for a particular local WSP design. Furthermore, if such a relationship was known it could be used to determine the area and number of ponds required.

The regression equation in Figure 6.18 reflects the optimal capacity of the FP at Khon Kaen City and it may be applied to neighbouring towns to assist in WSP design.

Unlike pond 1, which tended to display the most efficient natural treatment system, ponds 4 and 5 indicated no relationship between BOD loading and removal and even ponds 2 and 3 showed a lesser degree of correlation. This might be as a result of greater BOD loading in these ponds (exceeding 40-50 kg BOD/ha d) which was inappropriate for this WSP's ecosystem.

Another reason might be as a consequence of organic content (BOD) and nutrients (N and P) which were inappropriate for assisting the bacterial and algal relationship. It could be seen that pond 1 had a high contents BOD, NH_3-N and PO_4-P, and this might be the optimal amount of essential substrates needed for assisting bacterial and algae symbiotic relationships. On the other hand, the other ponds had low levels of these, therefore the ecosystem in those ponds was inappropriate for any biological mechanism.

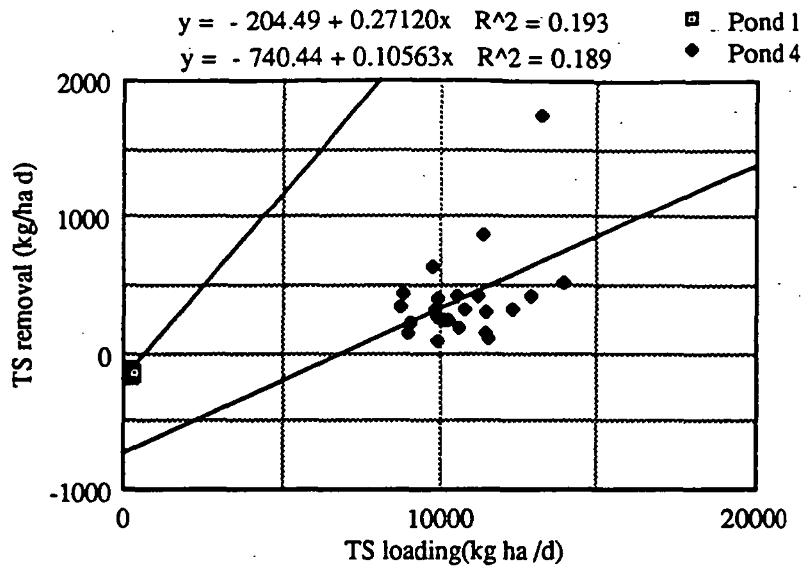
Figure 6.18 BOD loading and BOD removal relationship.



6.11.2 TS loading and removal

There was a correlation between applied loading and removal of TS in ponds 1 and 4, as illustrated in Figure 6.19 (pond 1, r 0.439, pond 4 r 0.434, P <0.05). Ponds 2, 3 and 5 had no significant correlation (r 0.164, 0.216, 0.076, p> 0.05).

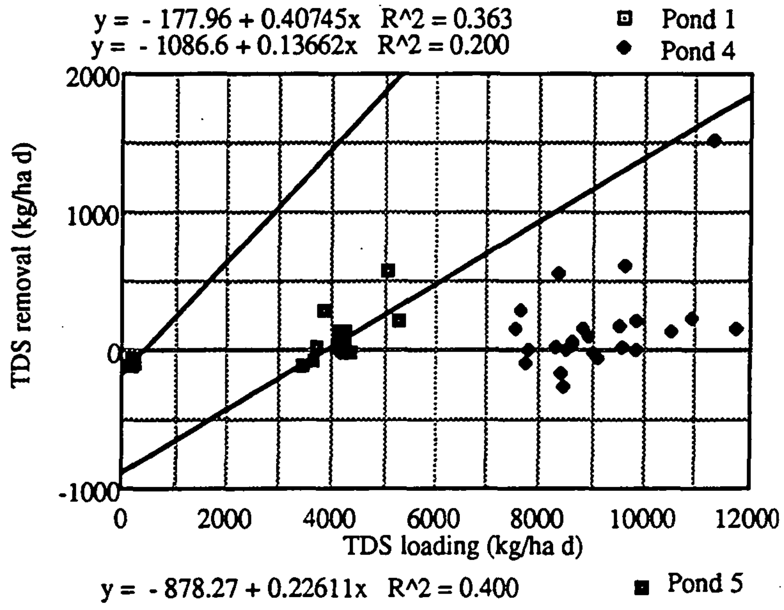
Figure 6.19 Relationship between TS loading and removal.



6.11.3 TDS loading and removal

There was a significant correlation between TDS loading and removal occurring in the first, fourth and last pond, as illustrated in Figure 6.20 ($p < 0.05$). From the second to the third pond, no correlation was found (r 0.164, 0.044, $p > 0.05$).

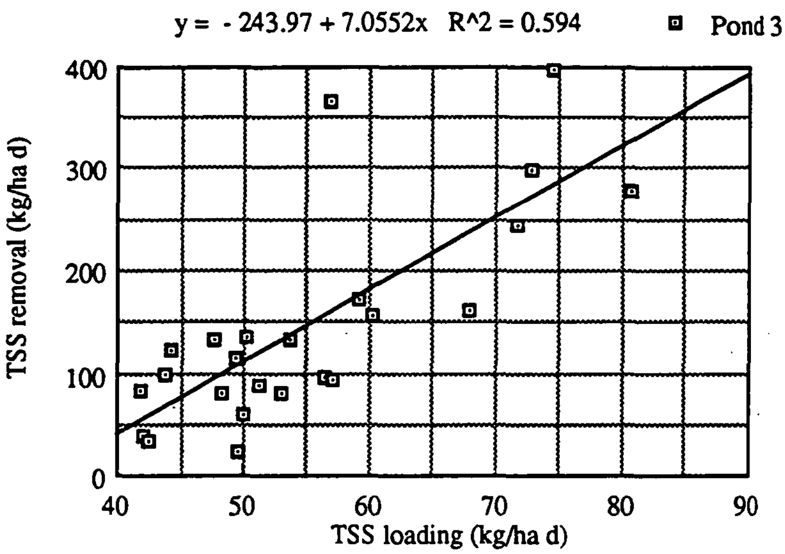
Figure 6.20 Relationship between TDS loading and removal.



6.11.4 TSS loading and removal

Positive correlation between TSS loading and removal was found occurring only in the third pond, as shown in Figure 6.21 (r 0.771, $p < 0.05$). There was no correlation of this relationship in ponds 1, 2, 4 and 5 ($p > 0.05$).

Figure 6.21 Relationship between TSS loading and removal.



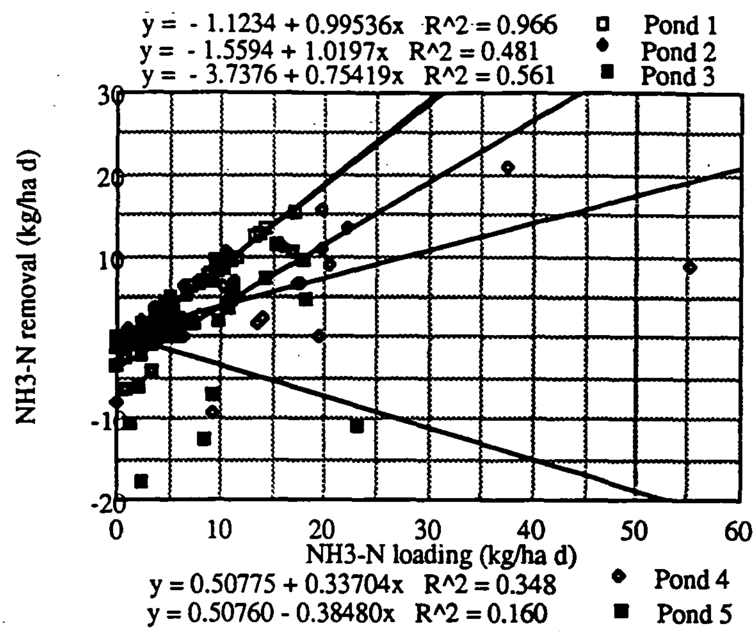
TDS and TSS levels in all ponds had higher values than in raw sewage. Only TS in the final effluent averaged lower levels than in the influent. It was difficult to draw a conclusion on these relationships, however, from the observation of ponds 1 and 4 as they appeared to have much more algal die-off floating up to the surface of the water column. Organic contents and nutrients limited the bacterial-algal symbiotic relationship and both those two living organisms died. According to this assumption, such bacterial-algal relationships might fail.

Another reason for no relationship of solid loading and removal might be the high levels of solid loading in the ponds. This might affect the capacity of the pond's environment to remove solid contents. However, currently, no research documents this failure of solid removal in WSPs.

6.11.5 NH_3-N loading and removal

NH_3-N loading and removal had a significant positive correlation in every pond ($p < 0.05$), as illustrated in Figure 6.22. Pond 1 had the highest degree of positive correlation (r 0.982, $p < 0.05$). Pond 5 showed a negative degree of NH_3-N loading-removal correlation (r -0.400, $p > 0.05$).

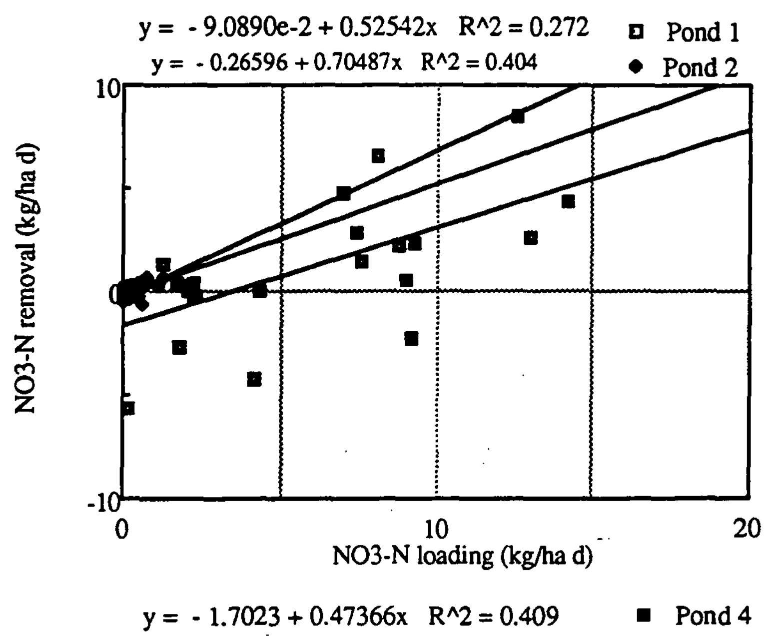
Figure 6.22 Relationship between NH_3-N loading and removal.



6.11.6 NO_3-N loading and removal

Ponds 1, 2 and 4, as illustrated in Figure 6.23, had a positive correlation between NO_3-N loading-removal ($P < 0.05$). Ponds 3 and 5 exhibited no significant correlation between NO_3-N loading and removal (r 0.316, 0.070, $P > 0.05$).

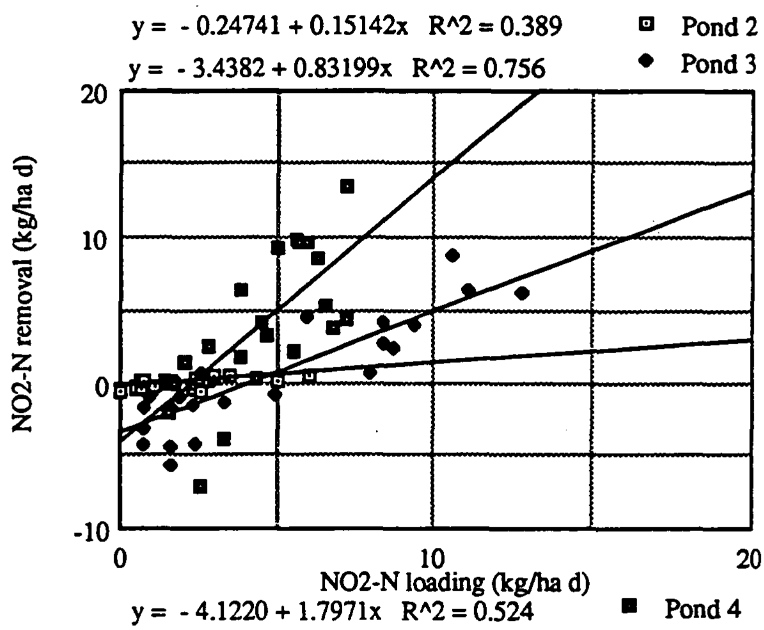
Figure 6.23 Relationship between NO_3-N loading and removal.



6.11.7 NO_2-N loading and removal

NO_2-N loading and removal had a positive correlation in ponds 2, 3 and 4, as shown in Figure 6.24 ($P< 0.05$). Ponds 1 and 5 indicated no significant NO_2-N loading-removal correlation (r 0.00, 0.190, $P> 0.05$).

Figure 6.24 Relationship between NO_2-N loading and removal.



The nitrogen loading and removal described in 6.11.5-6.11.7, especially the NH_3-N loading-removal relationship, could be explained by the uptake of NH_3-N by algae. The transformation of NH_3-N->NO_3-N by autotrophic nitrifying bacteria had a limited effect as NO_3-N levels were lower in every pond.

Pond 5 had a negative degree of NH_3-N loading-removal correlation. The NH_3-N level in pond 5 was higher than in the preceding ponds, where NH_3-N removal was -1.3. It was possible that algal cells settling towards the bottom, as displayed by TSS levels dropping in this pond compared to the preceding ponds, causing ammonification to occur. That meant a higher level of the NH_3-N eventually leading to a negative degree of NH_3-N loading-removal correlation.

Another reason might be the oxygenation condition in pond 5. Since this pond had the lowest level of O_2 compared to the other ponds and the oxidation reaction of NH_3-N needs about 4.5 parts of O_2 for each part of NH_3-N, pond 5's environment could not assist in oxidising NH_3-N completely, therefore there was still a trace amount of NH_3-N left.

The above two conditions, separately or together might, have caused the negative correlation between NH_3-N loading and removal occurring in pond 5.

For NO_2-N, the highest degree of correlation occurred in the second pond (r 0.868) and NO_3-N had the highest degree of correlation in pond 4 (r 0.639). According to Middlebrooks *et al.* (1979) both NO_2-N and NO_3-N, which were generated from oxidising NH_3-N, were found to be associated with a function of time. So, with a higher HRT as in the second pond, nitrification was expected to occur at a higher level. Consequently, the NO_2-N which was oxidised from NH_3-N could be expected to be further oxidised to NO_3-N. Thus, leading to a higher degree of NO_2-N loading-removal correlation.

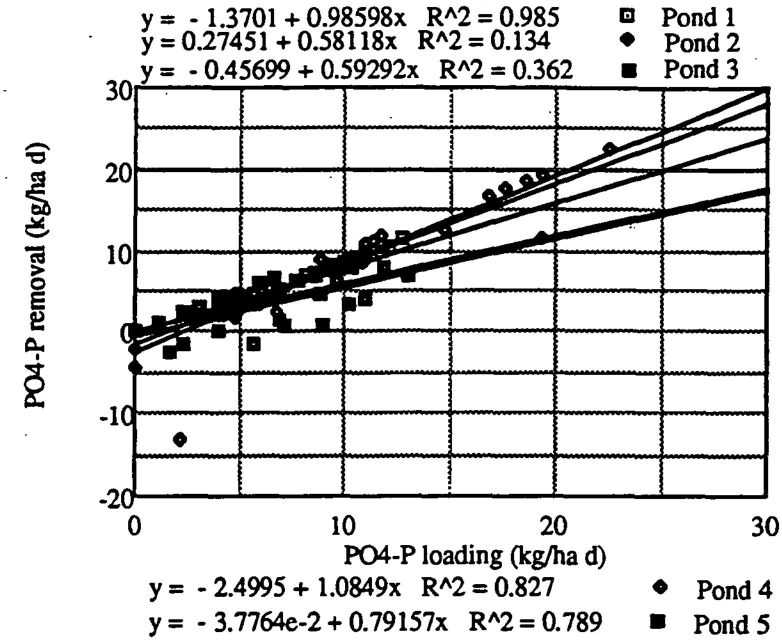
However, since there was a trace amount of NO_3-N existing in the ponds in an almost stable state, it was difficult to conclude which factors were the major causes of the loading and removal relationship of NO_2-N and NO_3-N. Specific investigation is required to explain this occurrence and currently there is no documentation associated with this nitrogen behaviour in WSPs.

6.11.8 PO_4-P loading and removal

All of the ponds showed a positive correlation of PO_4-P loading and removal, as illustrated in Figure 6.25 ($p < 0.05$). Pond 1 indicated the highest positive correlation (r 0.992, $P < 0.05$). With limited presence of certain microorganisms e.g. nitrifying bacteria species, PO_4-P showed more direct positive relationship with algae than that of NO_3-N. This might be as a consequence of some algae and other aquatic plants being able to fix nitrogen from the atmosphere, resulting in a low degree of correlation.

Thus, PO_4-P was found to be the most significant growth limiting nutrient for algae. All of the ponds exhibited a high degree of correlation, except pond 2 which had a lesser degree at r 0.366.

Figure 6.25 Relationship between PO4-P loading and removal.



6.12 Relationship between parameters

6.12.1 *BOD and other parameters*

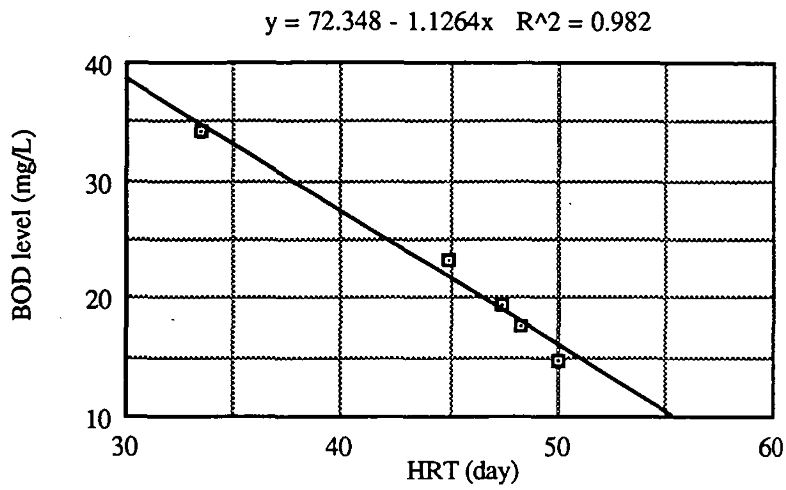
6.12.1.1 BOD vs HRT

With an increase in HRT in the WSP, it was found that BOD was reduced sharply, as shown in Figure 6.26 ($P < 0.05$). On the forty seventh day of HRT, BOD could be reduced to meet the country's required standard (20 mg/L).

A general conclusion can be made that the length of HRT appeared to be a major factor in decreasing BOD level. This was agreed by many authors such as Meron, Reebhun and Sless (1965) and Mara (1976) who advocated that HRT was the most important factor in reducing BOD.

At Khon Kaen WSP, it was found that an HRT of 47 days was required to attain the BOD 20 mg/L standard. This was longer than suggested in various documents mentioned in the literature review e.g. 25 days, Caldwell (1946), 7-20 days, Metcalf and Eddy (1972), 20-30 days, Ansari (1973) or even Reid (1982) recommended HRT for tropical countries at 17-33 days. Referring to Figure 6.26, it showed that it was quite impossible to attain the standard BOD level by lowering HRT from 47 days in this plant.

Figure 6.26   Relationship between BOD and HRT.



6.12.1.2 BOD vs COD

Plotting laboratory results of BOD vs COD levels, revealed that only pond 1 had a positive correlation between BOD and COD, as illustrated in Figure 6.27 (r 352, P <0.05). However, there was a low degree of correlation. It was possible that with the highest levels of COD (229.5 mg/L) and BOD (34.1 mg/L) in pond 1, the effect of the BOD-COD relationship was more apparent. For the other ponds, with lower levels of both parameters, no significant correlation appeared between these two parameters (P >0.05).

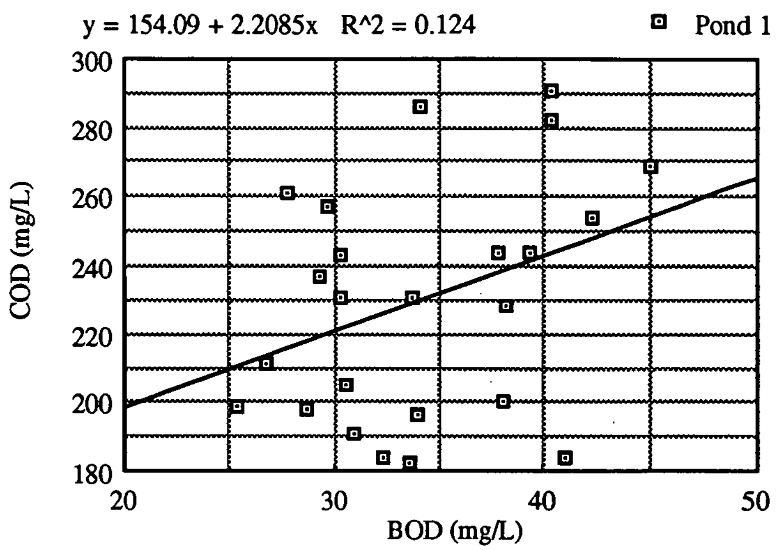
A relationship between BOD and COD was referred to by McKinney (1976). His investigation in the USA indicated a relationship between BOD and COD in some WSPs, where the value of BOD was 58% of COD with a 5% variation.

Khon Kaen WSP plant, from ponds 1 to 5, had percentages of 45.9%, 11.2%, 13.4%, 14.7% and 17.8% respectively. It was quite clear that only pond 1 had a value close to McKinney's, for the other ponds did not reflect his result.

As a result of the lower BOD and COD correlation found in the ponds, some important points needed to be taken into consideration such as (1) algal content was high from ponds 1 to 5 as reflected by TSS levels, thus COD level was consequently high (2) TSS, BOD and COD in the ponds fluctuated on each corresponding test-day (Appendix 1), resulting in a low degree of BOD and COD correlation (3) in fact the BOD test had limitations in that it was only a 5-day test, so during that time TSS (mainly algal content) would not be totally oxidised. This would affect the values of BOD_5, so the relationship between BOD and COD appeared to be varied and could not be accounted for.

Referring to the proportion of BOD to COD mentioned by McKinney, this proportion varied since the characteristic of domestic sewage was quite different from the industrial source. The variation in composition of domestic sewage was caused by the varying socio-economic characteristics of a community. The ratio of BOD:COD, as McKinney mentioned, could then not be applied in this context.

Figure 6.27 Relationship between BOD and COD.



6.12.2 *TS, TSS and other parameters*

6.12.2.1 TS vs BOD, COD and DO

There was no relationship between TS and BOD in any pond (r 0.114-0.294, $p > 0.05$). Explanation for this lack of relationship could be given by a number of factors including:

(1) TS was composed of biodegradable and non-biodegradable matters, whilst BOD consisted only of organic biodegradable matter.

(2) A major proportion of TS was TDS (see Appendix 1), and this TDS would not contribute to the BOD_5 test.

(3) TS and BOD were varying in levels of each pond.

(4) Volatile suspended solid (VSS), which represented the biodegraded wastes, was not included in the tests of this study. TSS could then be assumed to represent the biodegraded wastes. And from Appendix 1 it can be seen that TSS apportioned only a

small amount of TS. But, TSS constituted the main portion of the BOD_5 test. In other words, a large portion of TS was excluded from the BOD_5 test.

For all the above reasons TS and BOD would not be expected to have a correlation in the ponds.

TS and COD had a positive correlation in ponds 1, 4 and 5 (r 0.434-0.643, $P < 0.05$), as shown in Figure 6.28. No significant correlation was found in ponds 2 and 3 (r 0.187-0.327, $p > 0.05$). As mentioned earlier, the main portion of TS was TDS which was inorganic salt. This would not contribute to the COD value, so the correlation of TS and COD in the three ponds would be from TSS. Pond 1 showed the highest positive relationship of TS to COD. This might be as a consequence of pond 1 having a high content of all solids, such as algal cells, which could be oxidised chemically. However, this assumption will be confirmed by the relationship of TSS and COD in a later topic.

All of the ponds exhibited a high degree of positive correlation between TS and DO, as shown in Figure 6.29 (r 0.754-0.912, $p < 0.05$). The high positive correlation here in fact revealed that TSS, which was a portion of TS, showed a direct relationship to DO since it was mainly composed of algal cells. However, this point will also be confirmed in the TSS correlation topic.

Figure 6.28 Relationship between TS and COD.

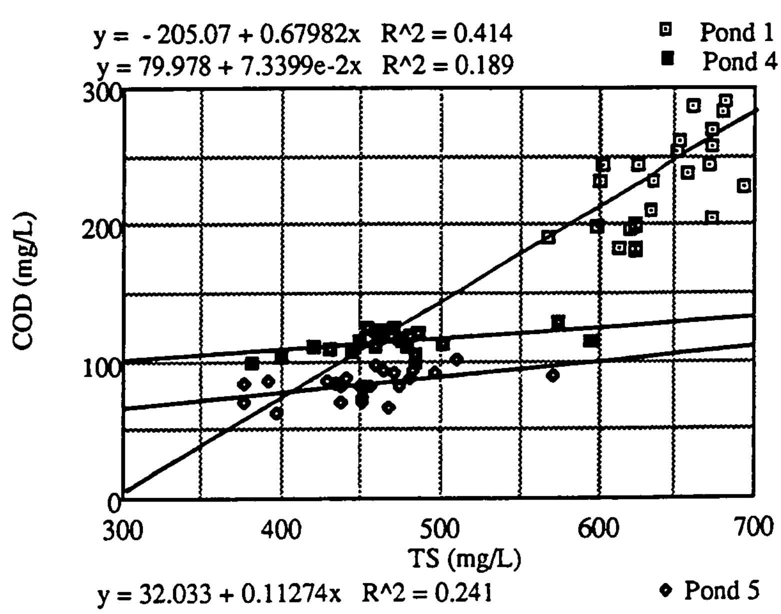
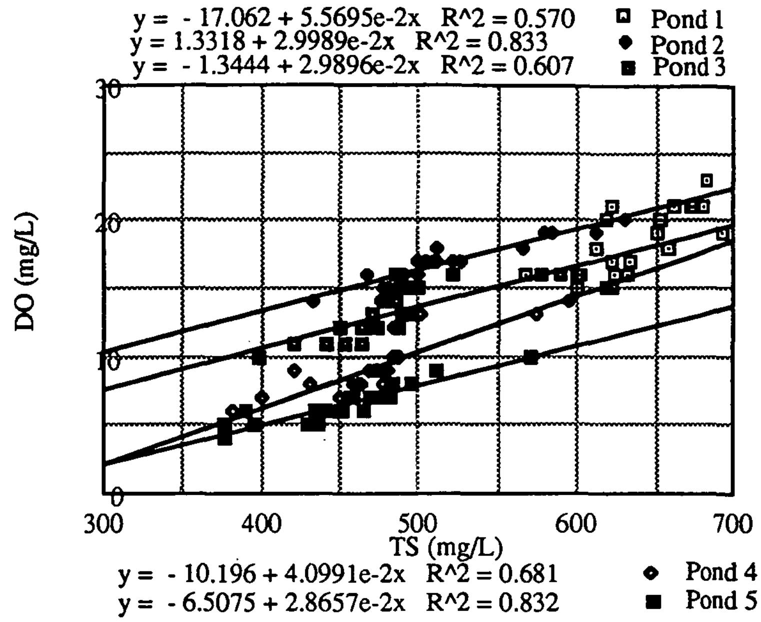


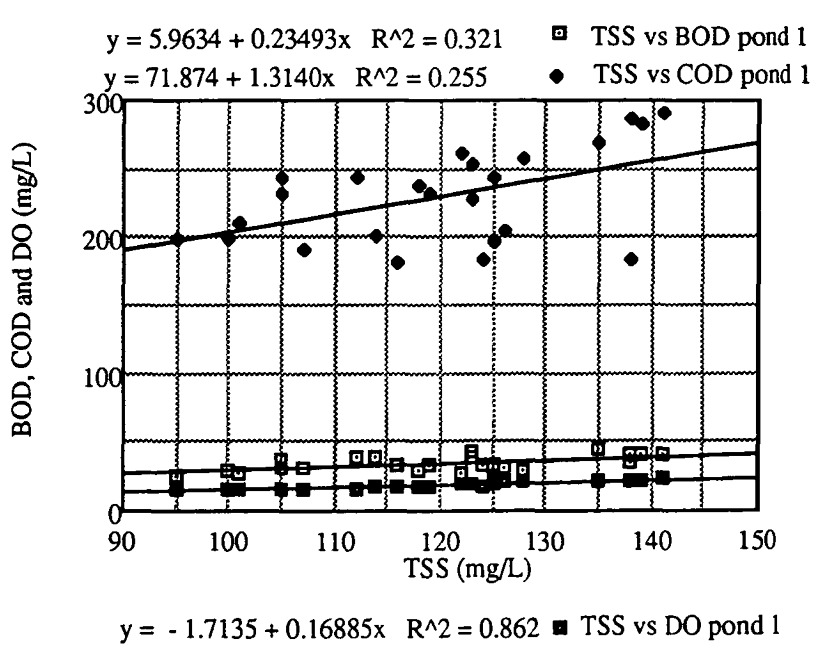
Figure 6.29 Relationship between TS and DO.



6.12.2.2 TSS vs BOD, COD and DO

All of the ponds exhibited a significant positive correlation of TSS with BOD, COD and DO (p >0.05) (see examples shown in Figure 6.30). However, TSS was found to have a higher degree of positive correlation to DO than to BOD and COD. To a certain extent, TSS was another measure of algal content in wastewater (Mara 1988), so this high correlation could reflect the oxygenation produced in the WSP from photosynthesis rather than by wind reaeration.

Figure 6.30 Relationship between TSS, BOD, COD and DO.



6.12.2.3 TSS vs NH_3-N, NO_3-N, NO_2-N and PO_4-P

TSS correlated negatively to NH_3-N (r -0.349 to -0.619, $P < 0.05$), positively to NO_3-N (r 0.403-0.604, $P < 0.05$), negatively to NO_2-N (r -0.380 to -0.546, $P < 0.05$), and negatively to PO_4-P (r -0.397 to -0.872, $p < 0.05$) (see examples in Figures 6.31-6.32). As TSS was mainly composed of algal cells, TSS levels were negatively correlated to NH_3-N and PO_4-P as a result of algal uptake of both elements.

TSS had a negative relation to NO_2-N. This resulted from a greater amount of algae existing in the ponds, leading to a greater degree of uptake of NH_3-N into its cell. Thus a lesser amount of NH_3-N remained to be oxidised to NO_2-N. A higher TSS could also contribute a greater oxygen supply to the pond so denitrification would not occur. Consequently whenever TSS increased NO_3-N also increased.

Figure 6.31 Relationship between TSS and NH_3-N, NO_3-N and NO_2-N in pond 1.

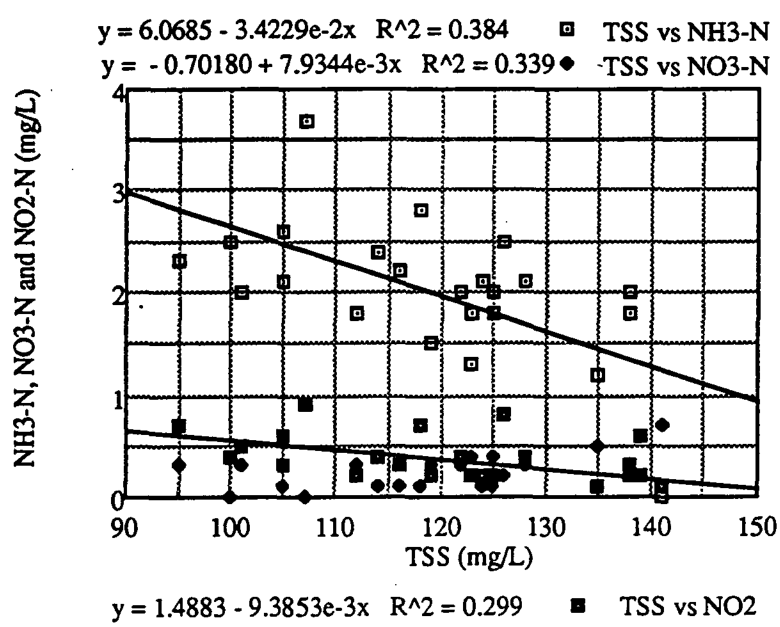
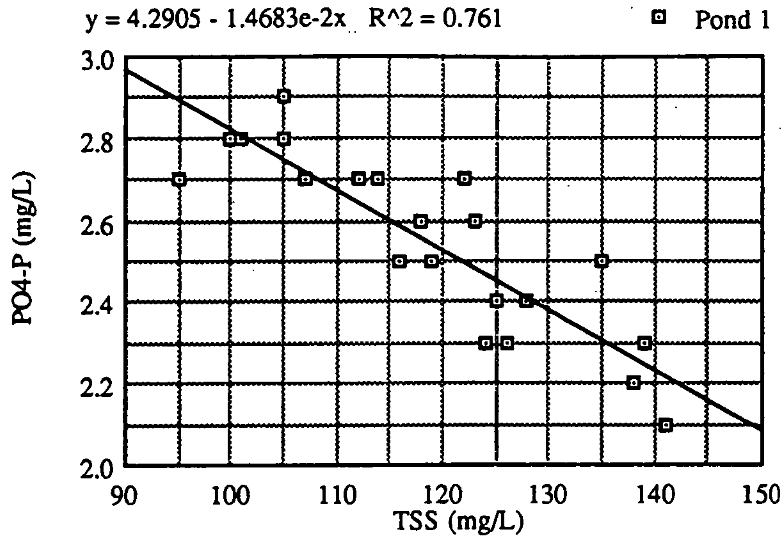


Figure 6.32 Relationship between TSS and PO4-P.



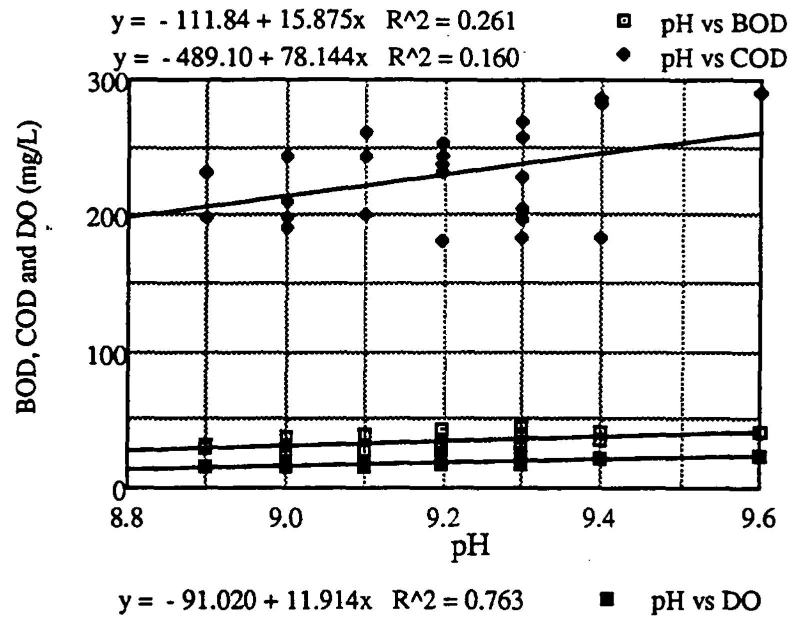
6.12.3 pH vs other parameters

6.12.3.1 pH vs BOD, COD and DO

The relationship between pH and BOD and COD had a positive correlation only in pond 1, Figure 6.33 (r 0.510, 0.400, $P < 0.05$). For pH and DO, there was a positive correlation in all ponds, Figure 6.33 (r 0.802-0.892, $P < 0.05$).

The positive correlation between pH and BOD and COD in pond 1, might be a result of the photosynthesis of algal cells. Algae consumes dissolved CO_2 for generating new cells, consequently BOD and COD were increased. Whenever dissolved CO_2 was exhausted, CO_2 would be drawn from bicarbonate, thus pH in the pond was raised. In other words whenever algal cells increased pH would also be raised. This occurred in a similar way to the positive relation of pH and DO too, so that as algal cells increased, DO increased.

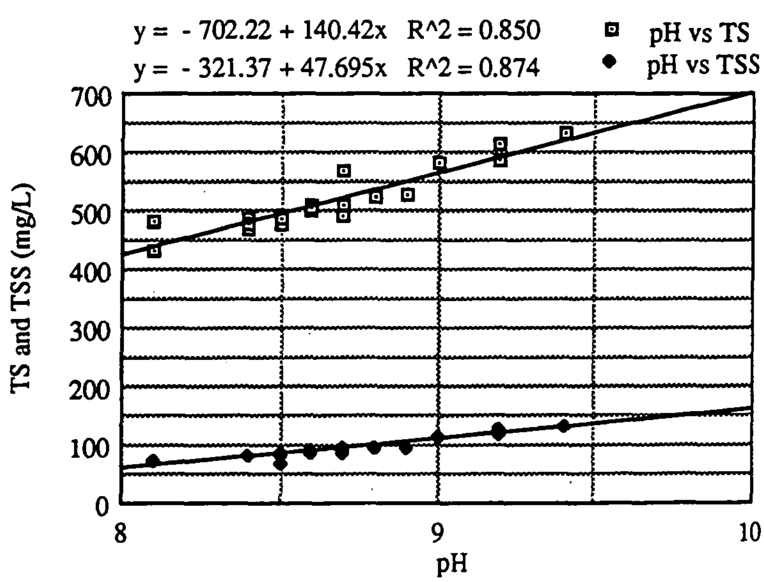
Figure 6.33 Relationship between pH and BOD, COD, DO in pond 1.



6.12.3.2 pH vs TS and TSS

There was found to be a positive correlation between pH and TS, TSS in all ponds, Figure 3.34 (P <0.05). These relationships clearly revealed that TS and TSS in the ponds were mostly algal cells which were photosynthesising and consequently raising pH levels in each pond.

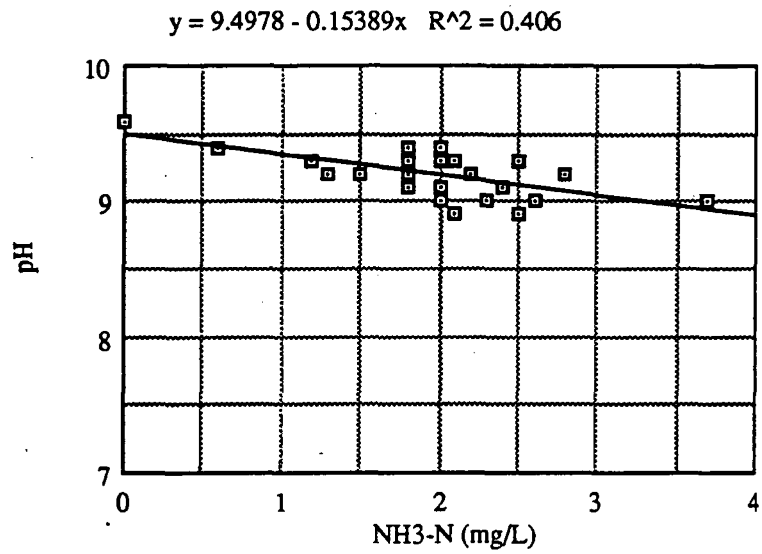
Figure 6.34 Relationship between pH and TS, TSS in pond 2.



6.12.3.3 pH vs NH3-N

Ponds 1 and 4 had a negative correlation between pH and NH3-N (r -0.474, -0.637, $P < 0.05$) (see Figure 6.35). Some ponds displayed a correlation of pH with NH3-N levels, confirming the fact that whenever pH was high NH3-N was formed rather than NH_4^+. Also, this might indirectly reveal that such a situation occurred as a result of a growing number of algal cells. CO_2 from bicarbonate was used by the algal cells, thus leading to raised pH, as mentioned earlier, involving carbonate equilibrium.

Figure 6.35 Relationship between pH and NH3-N.



6.12.3.4 pH vs TC and FC

There was a negative correlation between pH and TC, Figure 6.36 (r -0.407, -0.624, $P < 0.05$). Also, pH had a negative correlation to FC, Figure 6.37 (r -0.580, -0.821, $P < 0.05$). These two relationships were both found in effluent from FPs and MPs. According to Metcalf and Eddy (1972), the optimal pH for bacterial growth ranged from 6.5-7.5. Referring to pH in the ponds, it seemed that such levels of pH were not suited to bacterial growth, particularly *E. coli* which was more sensitive to pH. Also, this result conformed to Sebastian and Nair (1984) who found that higher pH led to higher *E. coli* mortality.

Figure 6.36 Relationship between pH and TC.

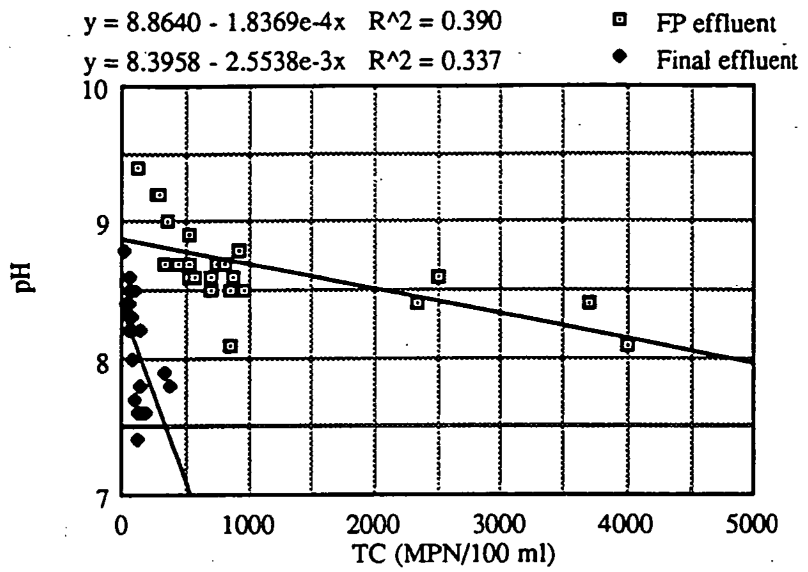
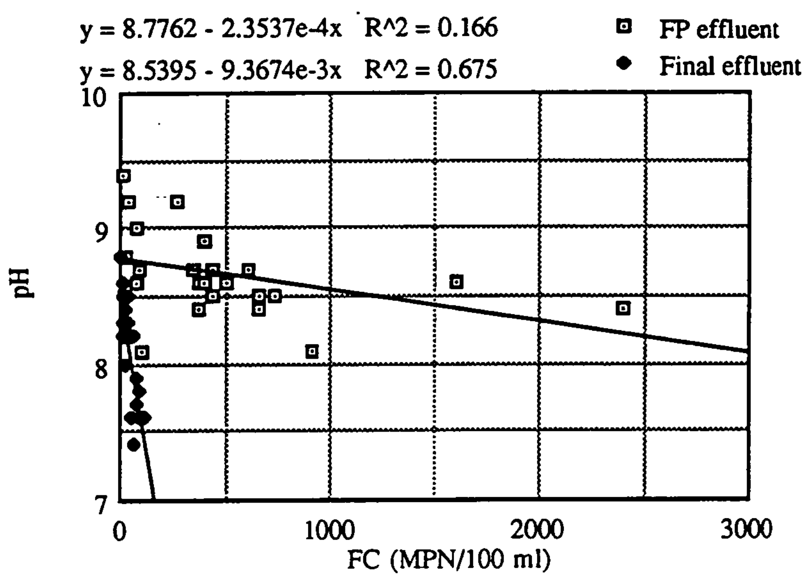


Figure 6.37 Relationship between pH and FC.



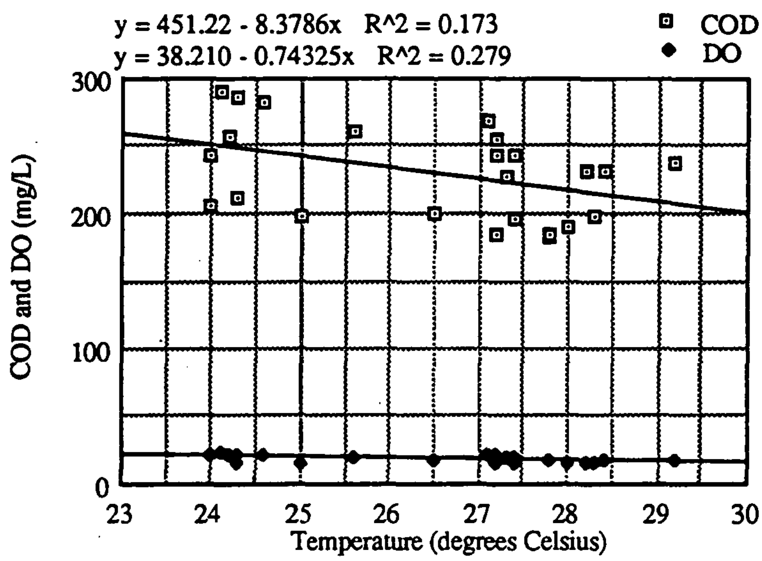
6.12.4 Temperature vs other parameters

6.12.4.1 Temperature vs BOD, COD, DO

Temperature was found to have a negative correlation to BOD in ponds 2 to 4 (r -0.352 to -0.541, $P < 0.05$). COD was negatively correlated to temperature in ponds 1 and 4, Figure 6.38 (r -0.415, -0.520, $P < 0.05$). DO and temperature were negatively correlated in ponds 1 and 4, Figure 6.38 (r -0.448, -0.528, $P < 0.05$). For the correlation between temperature and the three parameters above, even though such relationships did not occur in all ponds, it seemed that temperature had an effect in BOD reduction.

Some ponds demonstrated a negative relationship between temperature and BOD and COD which revealed in general that whenever temperature rose it would have the effect of increasing the oxidation rate in both biological and chemical reactions. Thus BOD and COD were detected at low levels. DO showed a negative correlation to temperature in some ponds. This meant firstly that when temperature was high O_2 in soluble form transformed to O_2 gases. Second, bacteria was activated when the temperature increased consuming more O_2, leading to O_2 occurring at lower levels.

Figure 6.38 Relationship between temperature and COD and DO in pond 1.



6.12.4.2 Temperature vs TSS

Temperature was positively correlated to TSS in all ponds (r 0.533-0.7.81, $P < 0.05$). This resulted from the effect of temperature on algal growth since TSS was mainly composed of algal cells.

6.12.4.3 Temperature vs NO3-N

For NO3-N, there was a significant negative correlation between NO3-N and temperature in ponds 1 and 5, as shown in Figure 6.39 ($P < 0.05$). Temperature and bacterial numbers showed no significant relationship ($P > 0.05$).

The negative relationship between temperature and NO3-N might be as a consequence of the temperature determining the degree of nitrification so whenever the temperature dropped NO3-N levels would lower.

As in Figure 6.40, it was found that temperature had a positive correlation to percentage of BOD removal (r 0.686, 0.728, $P < 0.05$). This would mean that to a certain degree whenever temperature increased percentage of BOD removal also raised.

Figure 6.39 Relationship between temperature and NO_3-N.

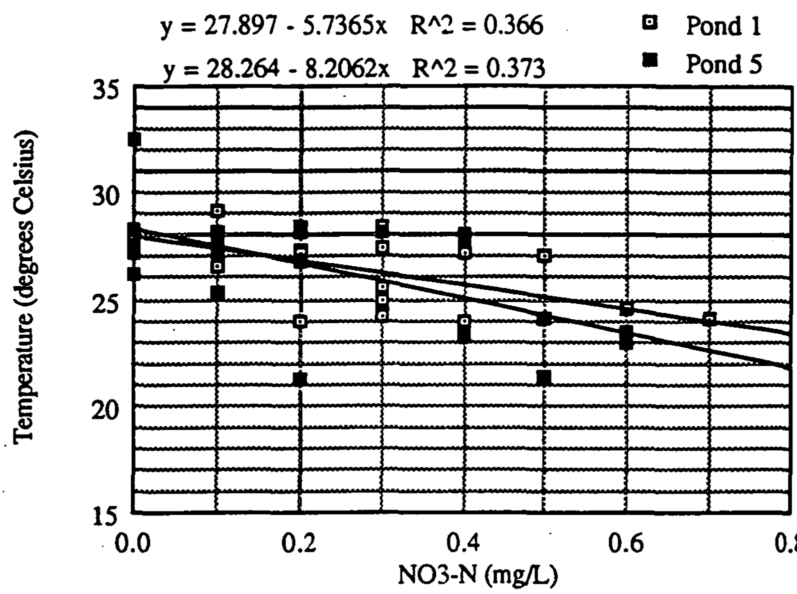
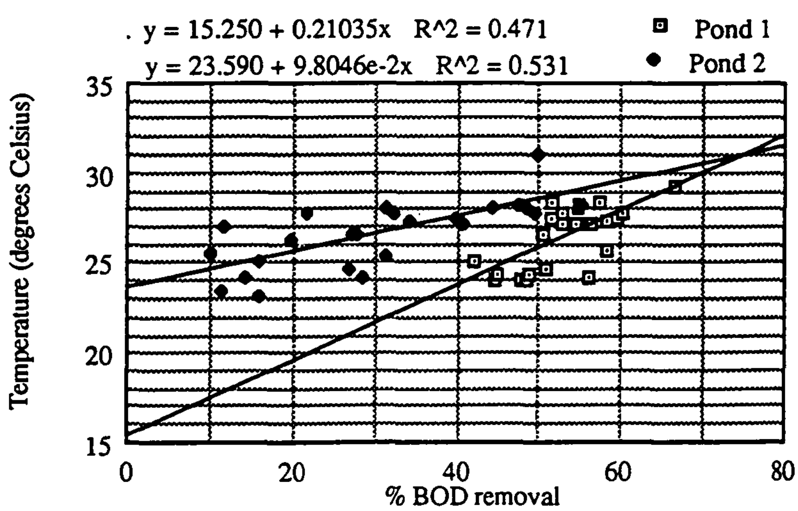


Figure 6.40 Relationship between temperature and percentage BOD removal.

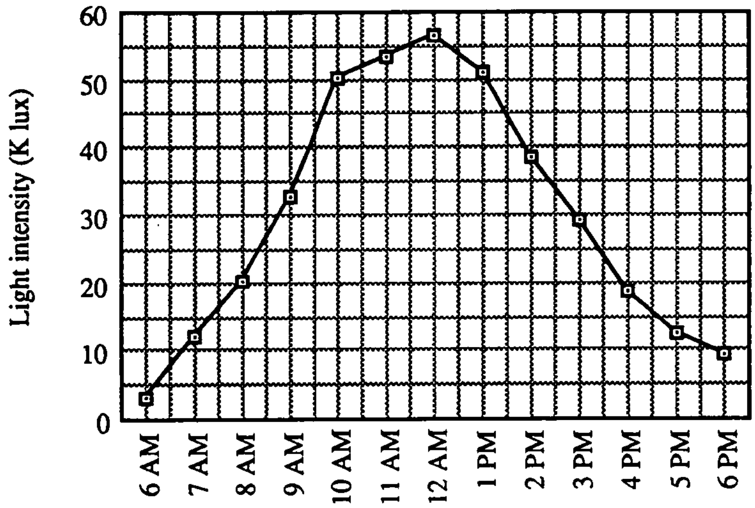


6.13 Daytime variation

Physical and biochemical factors including light intensity, temperature, pH, DO, PO_4-P, NO_3-N, NO_2-N, NH_3-N and H_2S were investigated so as to observe the daytime behaviour of these parameters inside the ponds.

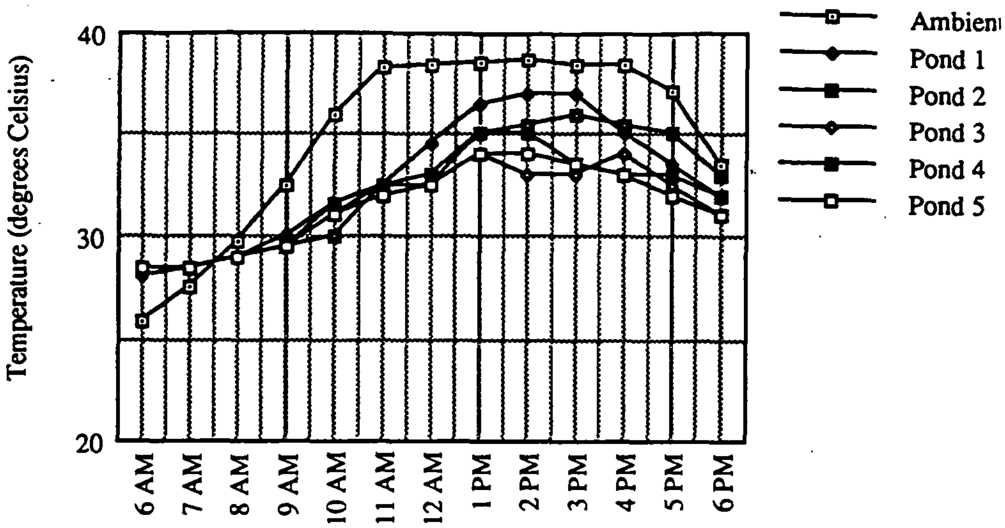
General circumstances on the test day (30th March 1993) were: the weather was clear, mild wind in the late morning and afternoon, the sky was clear and the sun was shining throughout the day. Light intensity was found at the highest peak at 11.52 am (around 12 am) with 56.4 K lux, as shown in Figure 6.41.

Figure 6.41 Light intensity at the Khon Kaen WSP (30th March 1993).



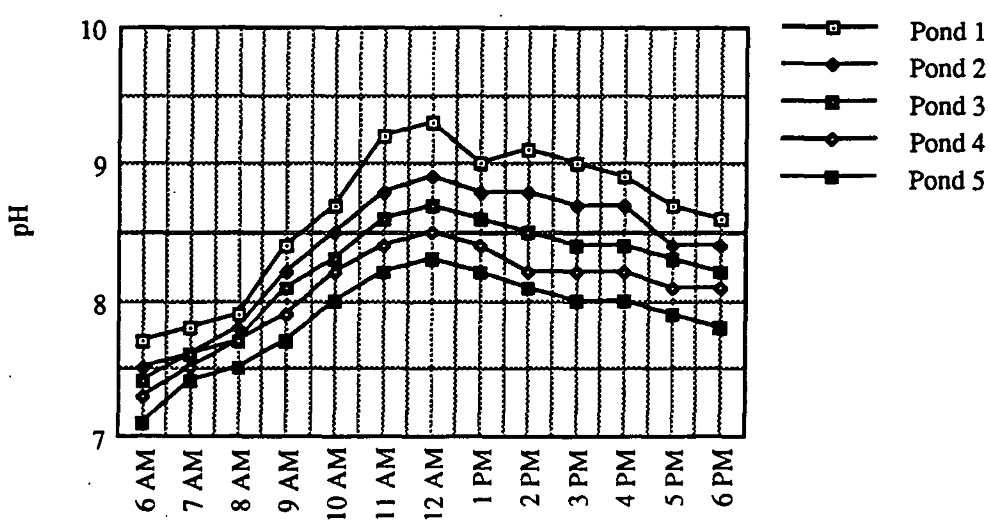
Ambient temperature in the vicinity of Khon Kaen WSP reached a maximum of 38.7 ^{0}C, at 2 pm, and the minimum was 25.6 ^{0}C, at 6 am. During 6-7 am, the temperature of all the ponds was higher than the ambient temperature. The temperature of most of the ponds reached its highest peak in early afternoon, around 1-2 pm. Average temperature of pond 1 was higher than the other ponds, as shown in Figure 4.42. From 2 pm, temperature in most of the ponds began to decline and ranged from 31-33 ^{0}C.

Figure 6.42 Hourly ambient and pond temperatures at Khon Kaen WSP.



The pH at 6 am in most ponds ranged from 7.1 to 7.7, pond 1 had the highest pH of 7.7 and the lowest was in pond 5. At 12 am pH of all of the ponds reached the highest peak, pH from ponds 1 to 5 were 9.3, 8.9, 8.7, 8.5 and 8.3 respectively. The pH pattern inside Khon Kaen WSP plant appeared to range from higher to lower values between ponds 1 to 5, as shown in Figure 6.43.

Figure 6.43 Variation of pH in Khon Kaen WSP during daytime.



At 6 am, pond 1 had the highest level of DO compared to the other ponds, which was 11.0 mg/L, whereas pond 5 had the lowest value at 4.1 mg/L.

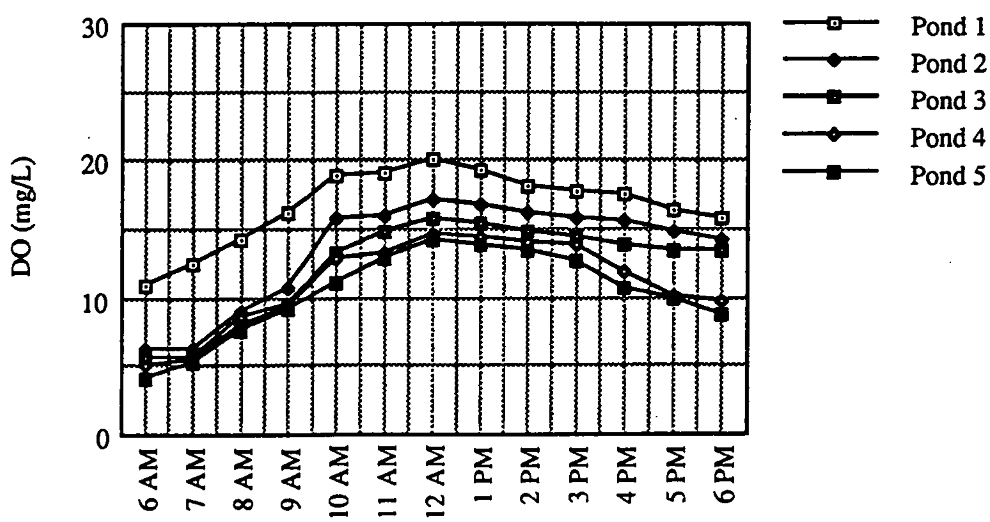
Also, the maximum DO level of the ponds occurred at 12 am in pond 1, 20.1 mg/L, and the minimum was in pond 5, 14.2 mg/L. Comparing DO levels among the ponds, shows that a pattern from higher to lower values of DO ranged from ponds 1 to 5, as illustrated in Figure 6.44.

DO levels increased from early morning and were at the highest levels in the afternoon, this was mainly due to light intensity and algal cell generation. When the light was more intense, photosynthesis by algal cells was more active, thus giving rise to DO at high levels. Therefore, the light and DO curves appear to be similar.

Such active photosynthesis by algae also made the pond condition alkaline. Since algae used dissolved CO_2 in excess of the supply of bacterial respiration, this led to CO_2 ionising from carbonate, and the pond became basic. The pH curve was also found to follow the light and DO curve.

Even the DO and pH curves appeared to have a pattern following the light curve, but the two curves were found to be different in each pond compared. This was largely due to the different levels of algal content in each pond. Pond 1 had the highest content (TSS), so it would have higher levels of DO and pH than the other ponds with lower algal content.

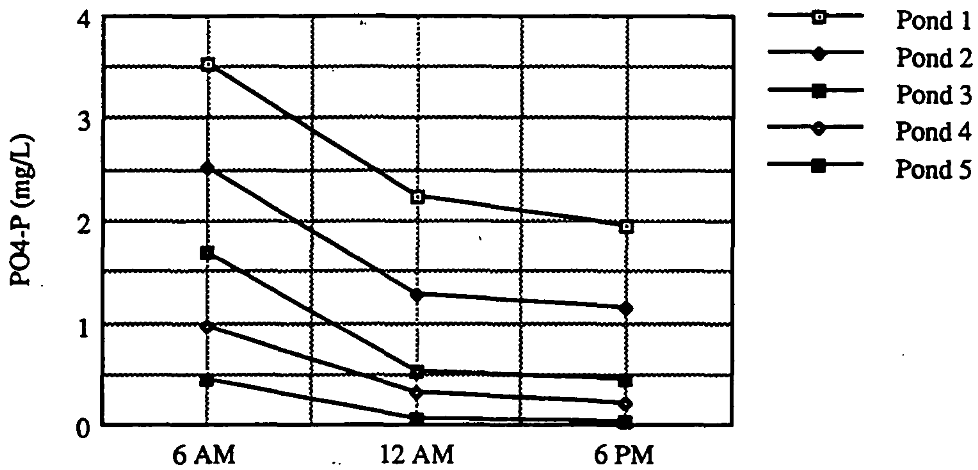
Figure 6.44 DO levels inside the ponds during daytime.



PO_4-P was at a higher level in the early morning in every pond. Pond 1 had the highest PO_4-P value compared to the other ponds, as shown in Figure 6.45. From midday to late afternoon PO_4-P levels dropped in every pond. The lowest PO_4-P level was in the fifth pond which was 0.037 mg/L at 6 pm.

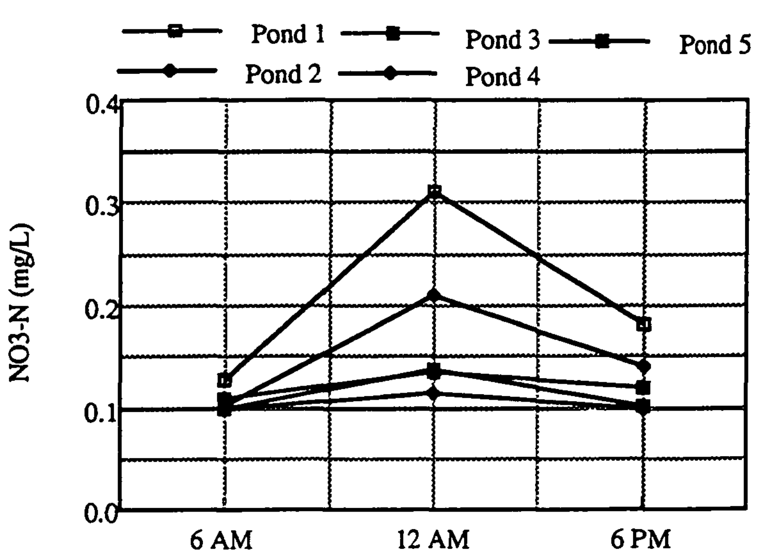
Lowering of PO_4-P levels occurred in the ponds as a consequence of more algal cell generation, from active day-light photosynthesis, leading to an absorption of dissolved PO_4-P by algae inside the ponds.

Figure 6.45 Daytime PO_4-P variation at Khon Kaen WSP.



NO_3-N was at a lower level in early morning and became higher at midday, Figure 6.46. Late afternoon, NO_3-N levels slowly dropped. The highest level of NO_3-N appeared at noon, this could be a result of excess oxygen produced by the algae inside the ponds, so that nitrification occurred (NH_3-N->NO_2-N->NO_3-N).

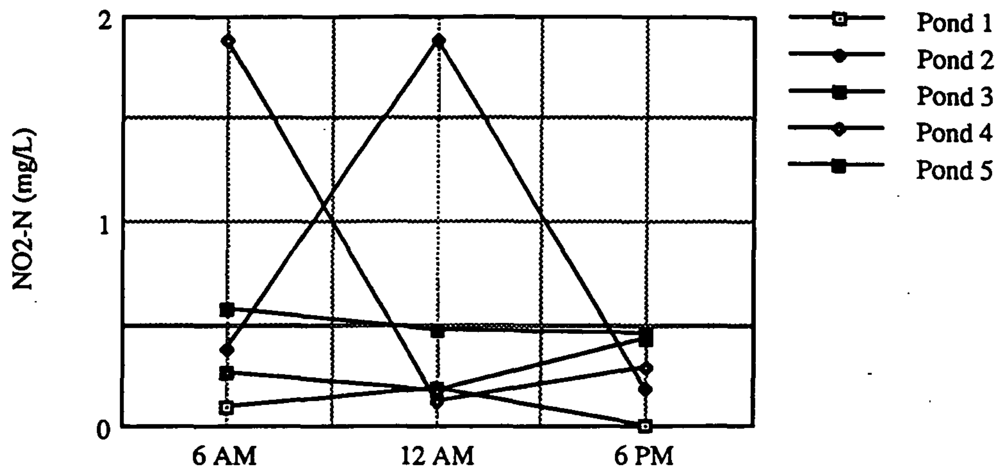
Figure 6.46 Periodic NO_3-N levels at Khon Kaen WSP.



Since NO_2-N was a transition element between NH_3-N and NO_3-N, and also involved a complex relationship of bacterial oxidation-reduction, NO_2-N levels shown in Figure 6.47 could not be explained by the author.

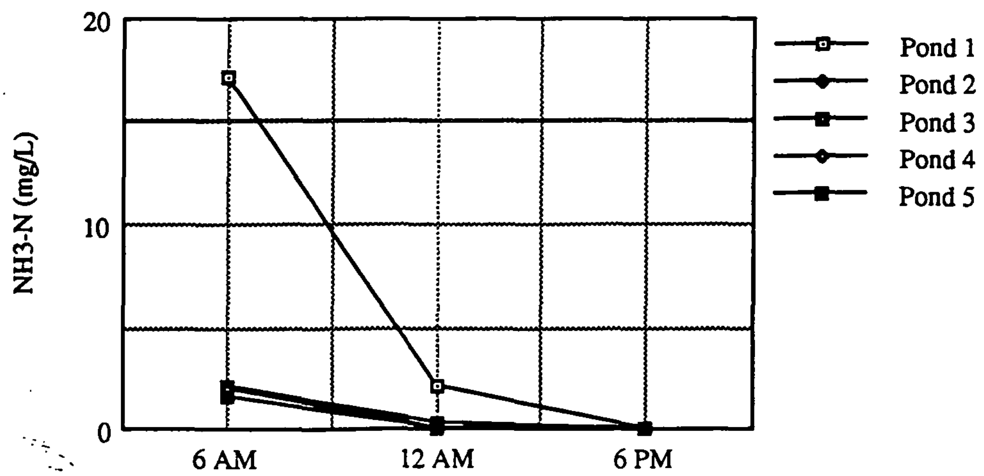
This would need more data associated with NO_2-N and the nitrogen cycle as a whole as well as physical and biochemical factors affecting NO_2-N levels.

Figure 6.47 NO_2-N levels inside the ponds.



NH_3-N values were higher in early morning and dropped by noon, until late afternoon when no value of NH_3-N could be read in the five ponds, as illustrated in Figure 6.48. The decline of NH_3-N levels from the morning till late afternoon was largely due to an absorption of the nutrient by algal cells while sunlight was prevailing. This curve could be referred back to the light intensity curve which was virtually opposite to the NH_3-N level curves.

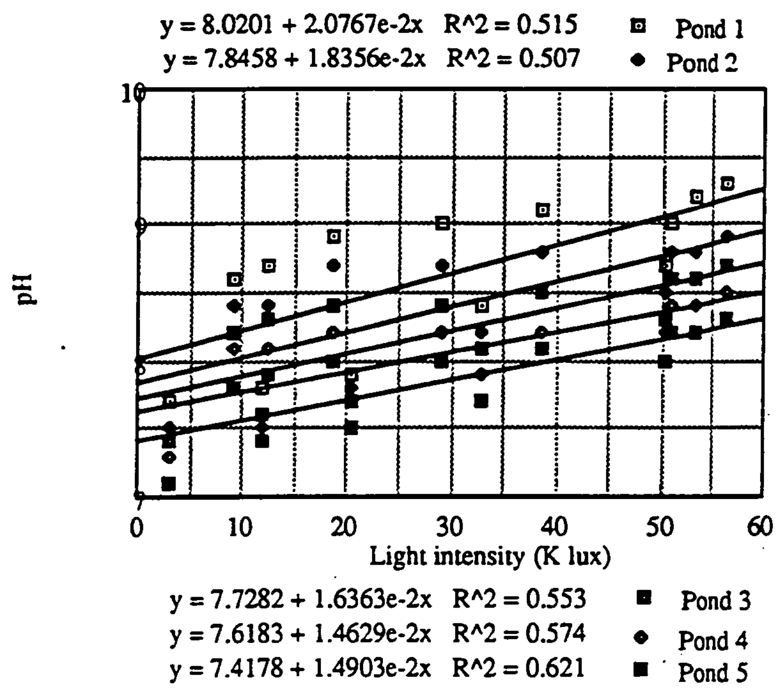
Figure 6.48 NH_3-N levels inside the ponds.



No H_2S level was detected in the daytime variation test. This condition revealed that H_2S produced from the pond bottom by anaerobic decomposition would be further oxidised by oxidising bacteria which was photosynthetic. Sulfate, which was the product of this reaction, would be reduced to SH^- by sulfate reducing bacteria using it as a cell constituent. So, H_2S could not be detected in the upper water column. Consequently, public nuisance caused by foul odour from the WSP could be attributed to algal floating mats not by H_2S.

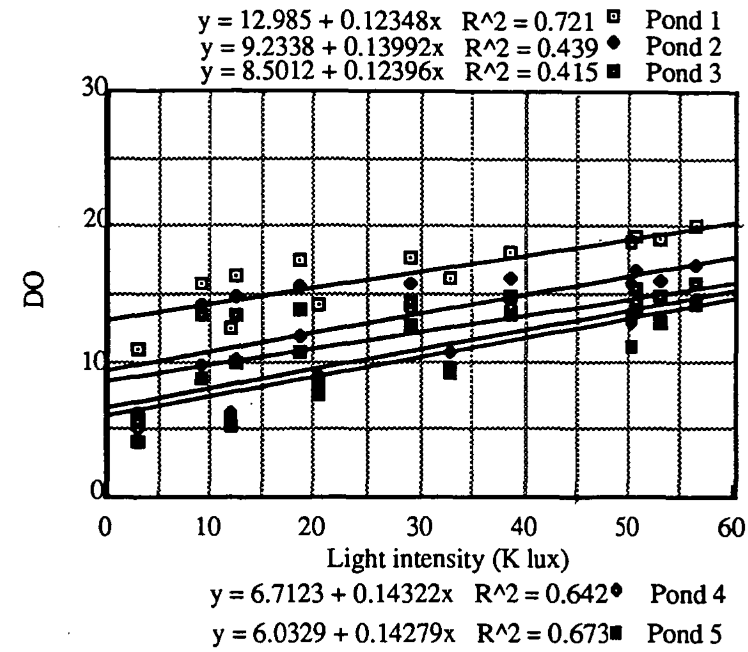
There was a positive correlation between light intensity and pH occurring in every pond (n 13, df 11, P <0.05). This was illustrated in Figure 6.49. An initial explanation could be that such a high pH occurred due to prevailing light and algal photoshynthesis, from which CO2 was removed at the dissolved phase therefore the hydrogen ion was decreased.

Figure 6.49 Relationship between light intensity and pH inside the ponds.



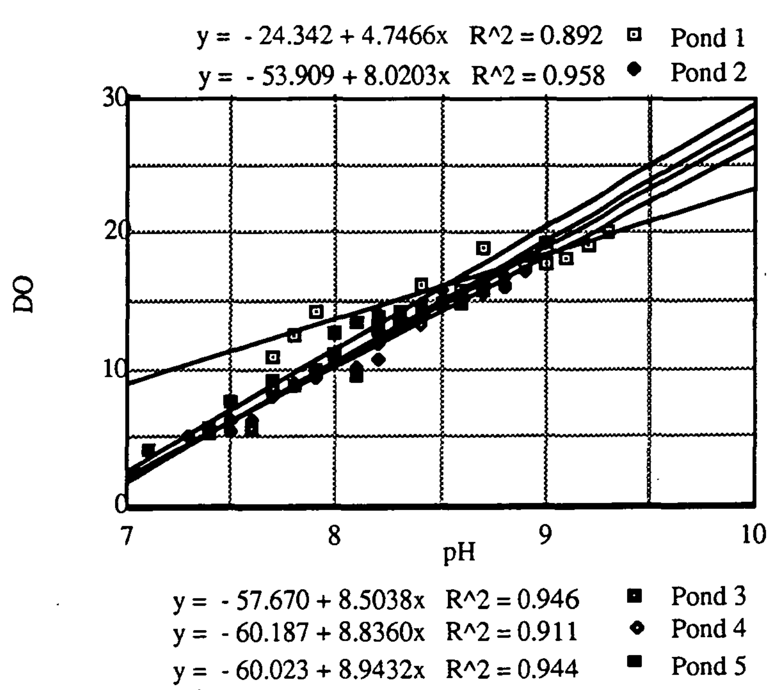
Whenever light intensity was high, there was a tendency for increasing DO levels inside the ponds (n 13, df 11, P <0.05). This could be shown in Figure 6.50. Algal photosynthesis with more light availability appeared to be a major source of increasing DO levels.

Figure 6.50 Light intensity and DO levels inside the ponds.



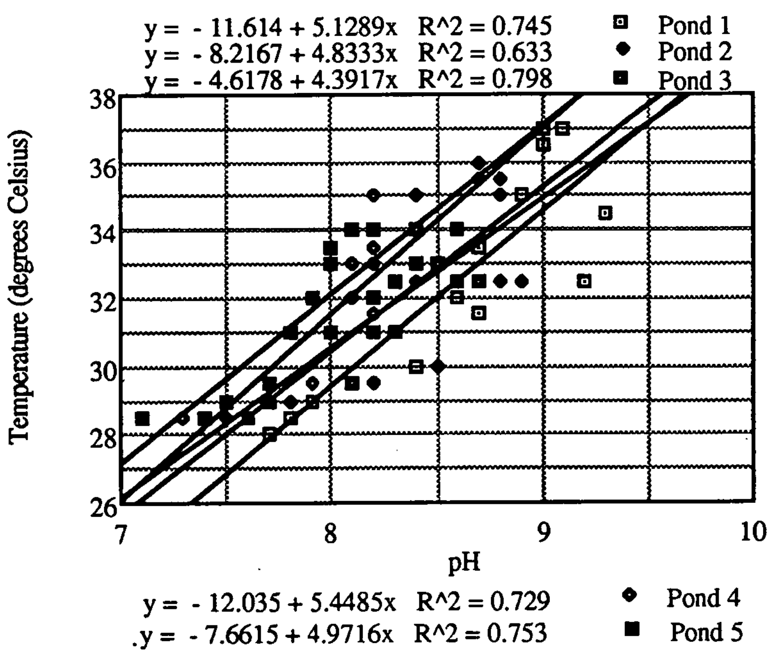
A relationship between DO and pH inside the ponds was quite positive, as shown in Figure 6.51 ($P < 0.001$). This seemed to indicate that the DO source was mainly from algal photosynthetic activities since pH was raised to a higher value when photosynthesis occurred, and so, higher DO levels were recorded.

Figure 6.51 DO and pH relationship inside the ponds.



Temperature and pH inside the ponds had a positive correlation, as illustrated in Figure 6.52 (P <0.01). Temperature was a very important factor in determining certain species of algae, bacteria and other aquatic organisms, as observed from this study. When the temperature increased there was an abundance of algal cells generated, especially *Anacystis* spp. During the daytime with active photosynthesis carried out these algal cells, higher pH values were recorded.

Figure 6.52 Relationship between temperature and pH.



6.14 Biota species associated with other considerations

6.14.1 Algal culture in the ponds

To find out what algal species inhabit all ponds of the WSP plant, a laboratory examination was conducted by a staff member of Khon Kaen University, Associate Professor Srisumon Sitathanee. Algal cultures were investigated in the Biological Laboratory, Faculty of Science with the provision of equipment and aids to the identification of algal species.

In the first facultative pond, there were three main groups of algal culture; these were green, golden brown and blue-green algae. The green algae were *Chlamydomonas* spp. and *Scenedesmus* spp., whereas the golden brown alga was *Navicula* spp. The blue-green algae were the largest most common algal species found dominating this pond, particularly *Oscillatoria* spp., *Spirulina* spp., and *Anacystis* spp. Of all algal species, *Anacystis* spp. was the most abundant in this pond.

In the second facultative pond, two groups of algae were present; the green and blue-green algae. Green algal species were *Pandorina* spp. and *Chlamydomonas* spp. Three species of blue-green algae were also found; these were *Oscillatoria* spp., *Spirulina* spp. and *Anacystis* spp. The blue-green algal species existing in this pond were similar to those in the first pond.

In the third pond, which functioned as the first maturation pond, only the blue-green algae existed. *Anacystis* spp. and *Spirulina* spp. dominated this pond.

In the fourth pond or the second maturation pond, green and blue-green algae were found. The blue-green algae were *Anacystis* spp. and *Spirulina* spp.; the green alga was *Scenedesmus* spp.

Only the blue-green algal species were found inhabiting the last pond. They were *Anacystis* spp., *Spirulina* spp. and *Synechococcus* spp.

In all of the ponds, the algal species which were most common and existing in greatest number compared to the others was *Anacystis* spp.

Given these phytoplanktonic species, there were a number of conditions found to be associated with algae culture in Khon Kaen WSP. These were as follows:

(1) Since there was a large number of *Anacystis* spp. found flourishing in every pond, offensive odour was produced in all ponds. There was another odour-producing algal species in pond 2 which was *Pandorina* spp. But since the number of the latter was minimal, the detected odour in this plant could be largely due to *Anacystis* spp.

The generation of foul odour which provoked public complaints, generally occurred during the summer months. The production of odour by blue-green algae conformed with the works of many authors such as Weigand (1983), Odegaard, Balmer and Hanaeus (1987).

(2) As the dominant phytoplanktonic species in all ponds, *Anacystis* spp., a micro-algae, was unable to pass through a membrane filter during solids testing. TSS values were mainly due to *Anacystis* spp. Figure 6.30 reflected that *Anacystis* spp. had a major role in producing O_2 inside each pond. Also, *Anacystis* spp. related positively to COD levels in the pond (Figure 6.30).

(3) With reference to TSS having a strong negative correlation with nutrients such as PO4-P in Figure 6.32, it could be deduced that *Anacystis* spp. utilised PO4-P and generated the new algal cells found thriving in every pond. So, the TSS test to some extent would indicate the number of *Anacystis* spp. in the ponds. Not only did they consume nutrients inside the ponds, *Anacystis* spp. had a major role in pH change as indicated in Figure 3.34.

(4) Comparing the colour of incoming raw sewage and the effluent from the plant, it was evident that the colour of the effluent was greenish whereas the raw sewage was colourless. The greenish colour was common for all pond effluents. This colour was mainly due to the presence of *Anacystis* spp. in all ponds as indicated in Picture 1.

(5) *Oscillatoria* spp. was also found flourishing in the second pond. This species is recently known as a nitrogen-fixing alga. As observed in pond 2, the filamentous-shaped *Oscillatoria* could be easily seen suspended heavily in this pond. Thus, pond 2 according to this existing species, was not necessarily dependent on NO3-N.

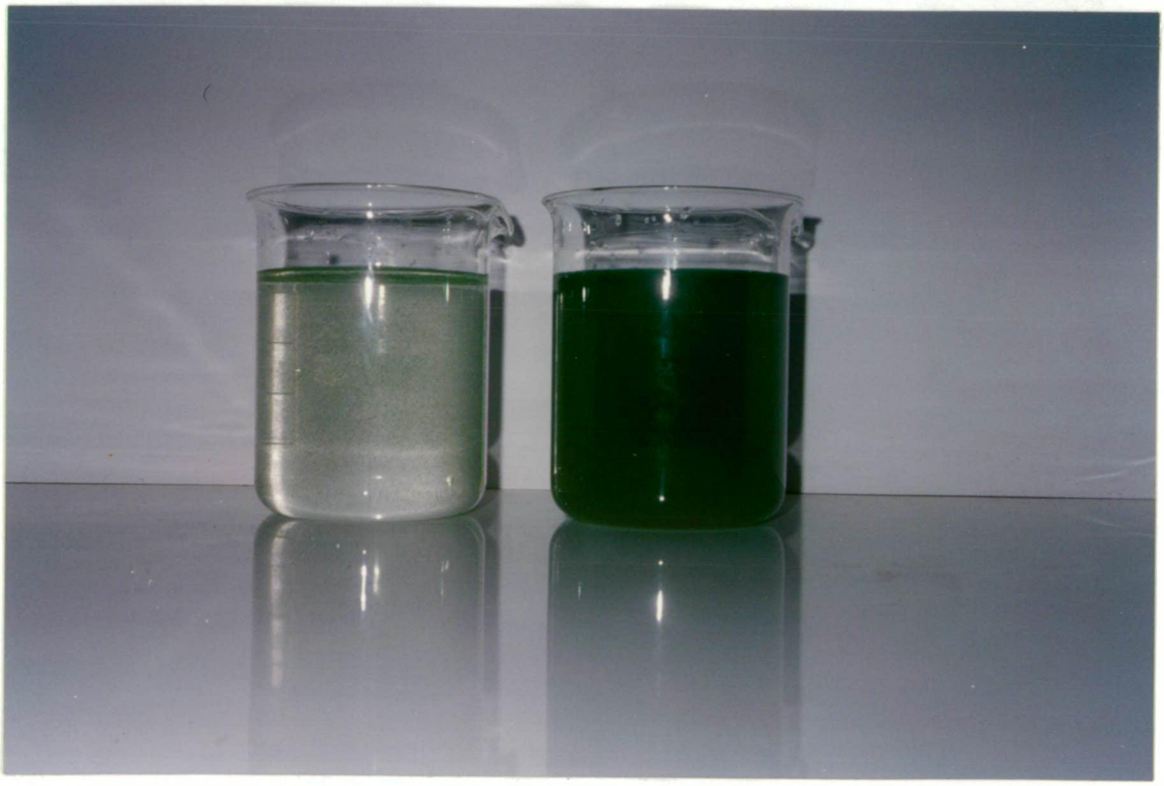
(6) By observing the ponds, it seemed that whenever the temperature of the pond liquid increased, the blue-green algae, especially *Anacystis* spp. could be readily seen as coccoid shapes suspended in the water column. With steady increase in temperature, these blue green algae seemed to adhere to each other eventually forming algal mats. Thus, the blue-green algae culture seemed to be temperature-dependent.

This algal behaviour of temperature-dependence was in agreement with the findings of Anderson and Zweig (1962) and Tariq and Ahmed (1981).

(7) The blue-green algae's behaviour related to temperature was observed in this study based on the ponds' daytime appearance. During the early test days, January 24th to February 12th, 1993, when the ambient temperature ranged from 25.1-27.0 ^{0}C, there were fewer blue-green algal mats in the ponds.

In contrast, from February 13th to the end of the field study, when temperature ranged from 29.2-32.5 ^{0}C commencing from 1:00 pm daily, there were more blue-green algal mats floating (Picture 2). The floating of algal mats persisted for 2-3 hours. Upon close examination, algal mats appeared as milky green patches having a strong foul smell. At this time, the water temperature was very high (up to 30 ^{0}C) and pH of the water was observed to be above 8.5. Whenever the temperature dropped, these floating algal mats would slowly disappear and settle at the bottom of the pond (Picture 3). Thus, it showed that the algal mats were sensitive to fluctuation in temperature.

Picture 1 Comparison of colour between raw sewage and final effluent (greenish).



Picture 2 Floating algal mats during a relatively warm day (31.0 ^{o}C).

Picture 3 Appearance of algal mats when the pond temperature is relatively low (22.1 ^{0}C)

(8) Regarding the observed effect of the wind, it was found that the direction of wind associated with pond location had an effect in aiding the settling of algal mats into the ponds. The wind could have lowered the water temperature which caused the algal mats to sink. An interesting point associated with the shape of the ponds was evidenced from ponds 3 and 4. During the study, there were numerous algal mats in these ponds, almost as dense as in pond 1. With lower levels of solids (algae) and nutrients (NH3-N, NO3-N and PO4-P) compared to pond 1, the pond shape appears to be a major factor in generating algal mats, primarily because these two ponds were irregularly-shaped (Picture 4) with smaller surface areas. Moreover, their location was not along the direction of the wind. Thus, the exchange of temperature between water column and air by wind induction was less in these two ponds. Therefore, the high temperature maintained in these ponds could have increased the number of floating algal mats.

Picture 4 The irregular shape of Pond 4.

(9) Like ponds 3 and 4, pond 1 had several patches of floating algal mats. This happened, not as a result of its location, but mainly due to its high nutrient content. However, comparing the settling time of algal mats between pond 1 and ponds 3 and 4, it was evident that algal mats in pond 1 had a shorter settling time (1-2 hours) compared to those in ponds 3 and 4 which took 5-6 hours. The location of pond 1 towards the direction of the wind could have assisted temperature transfer inside the pond, resulting in faster settling time.

(10) In relation to public nuisance, it was found that the wind had (1) the advantage of aiding algal mats to disappear, thus reducing the degree of offensive odour generated from these mats and (2) the disadvantage that it facilitated the transfer of offensive odour to neighbouring communities which resulted in complaints.

(11) Also, wind action was found to aid the movement of algal mats from pond surface to one corner of the pond. With location along the direction of the wind, algal mats in ponds 1, 2 and 5 appeared to be moved from spreading over the water surface to a particular corner. The accumulation of mats at one corner would help the plant personnel to remove these mats easily from the pond surface. In contrast with these ponds, the algal mats in ponds 3, and 4 were found spreading over the surface of the ponds which made the removal process difficult.

Apart from algal culture, zooplankton, protozoa and rotifers also appeared in this WSP plant. The protozoa such as *Stylonychia* spp., *Anisonema* spp. and *Pelomyxa* spp. were found. The rotifers were exemplified by *Proales* spp., *Epiphanes* spp. and *Filinia* spp. These zooplankton species might also have a major role in the pond ecosystem. However, investigation of this was beyond the scope of this study.

6.14.2 Fish culture in Khon Kaen WSP

Fish species existing in all ponds were *Oreochromis niloticus*, *Anabas testudineus*, *Fluta alba* and *Puntius gonionatus*. The two dominant fish species in great numbers were *Oreochromis niloticus* and *Puntius gonionatus*. The mollusc species dominating the ponds was *Viviparus* spp.

The abundance of some fish species in the ponds could be explained by the high nutrient content. Only four species were present in the pond which might mean that the environment was suitable only for these species which could tolerate the toxic condition of the pond.

According to Dart and Stretton (1977), *Anacystis* spp produced some toxins that were harmful to living organisms. These toxins were also present in the Khon Kaen WSP due to the abundance of *Anacystis* spp. Documents about *Anacystis* toxins and others are few, and most concentrate on human health issues such as algal toxins associated with the outbreak of gastro-intestinal tract diseases in humans from contaminated drinking water with toxins. As the major fish species in Khon Kaen WSP, the initial assumption might be that *Oreochromis niloticus* and *Puntius gonionatus* could better tolerate *Anacystis* toxins than the others.

However, in terms of human health hazards, this would be crucial. There were two main concerns about human health and these fish species in Khon Kaen. First, *Oreochromis niloticus* and *Puntius gonionatus*. were harvested as food and these species might have contained toxins harmful to humans. There is little current research on and related to this topic and no conclusion can be drawn from their publications. This would be an appropriate topic of study related to WSPs in the future.

Second, whenever these toxins were discharged into a receiving stream and that water body was used for human consumption, there would be another direct effect on human health. As Dart and Stretton (1977) and Mara (1988) documented, there are only few investigations on this topic. Even in Australia, a current outbreak of algal blooms in the Murray-Darling river in Sydney led to some research projects on toxins.

Khon Kaen City, in which the water supply plant was located upstream, was relatively safe from toxins. However, there were a number of cities located downstream in which the water source can be polluted by toxins coming from the WSP plant. Since there appeared to be no document on these toxins released by *Anacystis* spp.--- about how long it would take for these toxins to degenerate, to what degree of dilution these toxins would still be safe for human consumption, or what concentration or strength of these toxins could be tolerated by the human body --- all of these still require further study in relation to WSP algal discharges.

As an economic advantage, fish reared in the WSPs may be viewed as a source of protein for the local people. Based on a ten-week observation, approximately 50 kg of fish per day were harvested from all five ponds. This was worth $A 77 per day. Most of the fish harvested, as fishermen claimed, were brought home and cooked as food. About 150 people belonging to 37 families depend on these fish, harvested throughout the year without having to buy from the market.

However, though the fish in the ponds functioned to control the number of algae (interrelating to other pond biomass), currently there is no stringent measure to regulate the harvest of fish from these ponds. The City Council still considered this fish culture as an added value to the wastewater plant provided that it benefited the people. Fortunately, according to an agreement among fishermen on this wastewater plant, fish was harvested only at a length of more than 15 cm. So a number of fish of smaller size were left behind to function in the pond ecosystem.

There are a number of studies showing that fish culture could reduce TSS levels effectively (McGarry 1982, Pescod and Mara 1988, Yhdego 1992). Since TSS levels discharged from this WSP normally could not meet the effluent standard, measures should be developed in relation to fish rearing and TSS control.

The area of research involving fish culture and WSPs is currently being explored by many authors. However, some questions still remain, such as: what would be the appropriate fish number in the ponds? and what fish species would be effective in controlling a particular WSP area? Reference to different authors could not imply which fish species would be more appropriate. Most of the research done is on fish bio-mass increase in the ponds.

Some fish species were documented to have significance in reducing TSS such as *H. molitrix*, *A. nobilis* (Henderson 1978) and *T. aureaus* (Stickney and Hesby 1978) However, with different locations of WSPs, fish species should be considered according to the local ecosystem.

Given this, there should be an investigation of fish rearing associated with TSS removal in this plant, since this plant discharged high levels of TSS, above the effluent standard. Another group of future fish culture investigations should be about the association between algal toxins deposited in fish tissues and human health resulting from consumption of fish from WSP. Further research is required into the application of fish culture to any WSP, and its impacts on human health, in general.

An interesting phenomenon related to fish culture which occurred during this study was that there seemed to be a close relationship between fish behaviour and pond temperature. The fish were likely to appear near the upper layer of the ponds in the afternoon whereas in the morning and late evening they were near the middle or to the bottom of the pond. This might imply an effect of temperature and dissolved oxygen level to fish in the pond.

During the afternoon, the water temperature was very high and also the amount of dissolved oxygen decreased towards the bottom. Coincidentally at that time, the algal mats were floating and covering the pond surface, therefore DO level dropped at the deeper layer, so fish had to move towards the surface of the pond. On a very warm day during the field study, it was found that some small-sized fish (3-4 cm) died. To create an effective ecosystem, the removal of TSS from the WSP seemed to be complicated by its association with the fish culture problem. Future studies on aquaculture in relation to other biochemical factors in WSPs need to be initiated.

6.14.3 Bioculture along the embankment earth

In the three maturation ponds without concrete slab linings, aquatic weeds were found growing profusely. These weeds were growing along the bank creating a bio-community which included insects and amphibians.

Water hyacinth (*Eichhornia crassipes*) and common duckweed (*Lemna* spp.) were found commonly growing on the pond embankment. Water primrose (*Jusiaea repens*) inhabited the lower bank and was extending into the ponds. This is similar to Water paspalum (*Paspalum scorbiculatum*) which also stretched its roots into the dyke earth.

It was found that aquatic weeds growing on the bank created operational problems for the plant personnel. Removal of weeds was done every month and was found to take much time, on average one week per month. The plant operators revealed that during the rainy season, these weeds extended rapidly into the ponds covering one-third of the pond surface. This happened due to the absence of concrete slab lining on MPs' banks. Since the first two FPs were lined with embankment concrete slabs, this problem was minimised; no aquatic weeds and insect communities were found around these ponds.

Molluscs like *Bithynia* spp. were also in great numbers in the MPs. On the surface, water striders *Gerris* spp. were often found existing in both FPs and MPs. Damselfly nymphs, *Lestes* spp., were seen in great numbers in MPs. Shrimps, especially *Mysis* spp., also inhabited the ponds, especially MPs.

6.15 The WSP's structure as related to pond performance

The different percentages of BOD reduction in the five ponds given in 6.10 seemed to be related to pond area. In fact, being restricted to depth of 2 m, then to increase pond area increased HRT, since the degree of BOD removal was dependent on HRT. Pond 4 with the least area and thus having the least HRT also received the highest BOD loading and had the lowest BOD removal. So, pond area determining the HRT could be considered as one of the main factors needed to be considered as it influenced pond performance in terms of BOD removal.

Considering the current situation in this plant in terms of pond area and BOD loading, it seemed that ponds 3, 4 and 5 were designed without concern for BOD loading. As the heavy BOD loads occurred in ponds 3 to 5, these ponds then had a lower BOD reduction. Since these ponds had irregular shapes with one small sharp corner, it was found that there was thick, adhering debris at the corner which impeded passage of liquid into the next ponds. Because of that high load and inappropriate shape, all of the liquid waste in ponds 3 and 4 appeared to move with difficulty into the next ponds.

Plant operation difficulties in summer months, such as clogging accompanied by a strong foul odour, occurred in these ponds as reported by the plant operators. Even during the time of this study, that situation was observed. A great amount of debris was found in these two ponds. Thus, liquid movement inside and between the ponds was impeded. This would mean that the designers aimed ponds 3 to 5 for bacterial removal only, without concern for other factors affecting pond performance.

Referring to Figure 4.2, the Khon Kaen WSP was designed in a series pattern. However, this plant was arranged inappropriately when compared to the conventional WSP series model. This was because the conventional series, exemplified by Figures 3.1 to 3.5, had to take into account (1) BOD loading in each pond which also indirectly determined pond area, (2) shape and other structures such as embankment lining, location of the pond towards the wind and features that could reduce clogging, maximise liquid mixing and enhance liquid movement between ponds and encourage temperature transfer.

The appearance of Khon Kaen WSP, particularly ponds 3 to 4, did not agree with the conventional concepts of series WSP. This study was unable to determine that the Khon Kaen WSP, as a whole, was of the series type. The incomplete structure of this plant led to the occurrence of many problems in the plant as mentioned earlier.

The dimensions of the ponds were compared to Figures 3.6 to 3.11. Mara (1976) proposed a model emphasising separate aerobic and anaerobic zones, as in Figure 3.6. It was found that even when Khon Kaen FPs did not conform to this structure, the FPs' performance was measured in terms of efficacies in BOD and nutrient reduction.

Both Mara (1976), Arthur (1983) and Hess (1983) recommended that the inside slope of the FP had to be 1:3; Khon Kaen FPs had a slope of 1:2. This was found to have a significance in terms of causing bank erosion. Every pond in this plant faced the problem of embankment erosion which shows that such a slope of 1:2 was inappropriate. It was too steep so that as a result, bank erosion occurred. The concrete slabs were also found displaced and sliding into the pond liquid (Picture 5). Thus, the embankment slope suggested by the above authors would be more appropriate in preventing such problems.

Without concrete slab linings, the Khon Kaen MPs had a serious problem of bank erosion. Even if these ponds were made of compacted clay, this would not help due to wave action, especially during the rainy season. It could be observed that the three MPs had received some soil, and had become shallower (Picture 6).

Also, as a result of the absence of pre-cast concrete slabs, heavy vegetation became another serious problem occurring in the MPs compared to the FPs.

Weeds were growing profusely at the MPs' embankment whereas the FPs had fewer weeds. There was only planted grass in FPs which provided protection from dyke erosion.

Picture 5 Concrete slab lining eroding into the ponds.

Picture 6 Bank erosion which caused pond shallowing.

The interconnecting structure of the Khon Kaen WSP which allows liquid to pass through the ponds, is similar to that of Mara (1976) in Figure 3.8. It operates efficiently and there was no reported problem regarding this inlet-outlet structure.

The inlet pipes of Khon Kaen WSP which distribute raw sewage from the distribution box to the first FP are similar to those in Figure 3.9 (Arthur 1983). The presence of two inlet pipes extending separately to almost the centre of the first pond was found to enhance distribution of incoming raw sewage and help liquid mixing inside pond 1.

With all the ponds of Khon Kaen having the same depth of 2 m, it was found that this depth was appropriate. With regard to the pond performance, especially pond 1, BOD and nutrient levels could be reduced significantly as those previously mentioned. However, in other ponds with similar depth, there was lower efficacy in reducing those parameters. To achieve higher efficacy in the pond performance seemed to be less dependent upon depth but several factors as mentioned earlier.

Associating the factor of depth with vegetation growth is not an absolute conclusion. As Dinges (1982) suggested, three feet (0.9 m) was the most appropriate depth to prevent vegetation. This seemed to be irrelevant with reference to this plant. To prevent vegetation flourishing in any WSP, the factor associated with embankment lining should be considered. Though Mara (1982) documented that a depth below 1 m could not prevent weeds from growing, with the Khon Kaen MPs (2 m depth), such occurrence of heavy weeds shows that the depth was less related to vegetation growth.

In summary, the depth factor associated with pond performance and vegetation should also be based on geographical determination and other local conditions. For instance, recent research of Silva, Mara and Oliveira (1987) found that even when increasing the depth to 2.2 m, one Brazilian WSP could perform with very good results for BOD removal and other parameters.

6.16 Operation and maintenance modes

6.16.1 Operation and maintenance costs

Tables 2.1, 2.2 and Figure 2.1 indicated that WSP had the least construction cost compared to other systems. The Khon Kaen WSP also complied with this. The construction cost was $A 7.78 per capita or $A 40.46 per household (1989).

A figure of operation and maintenance (O&M) costs of WSP in Thailand by Rattanasuk and Grinsukon (1981) was $A 0.026 per kg BOD per day, this cost when adjusted to 1992 prices was $A 0.047 per kg BOD per day. The Khon Kaen WSP, with a 584.8 kg BOD per day figure in 1992, would have an estimated total cost of O&M $A 10,032.24. Nevertheless, the current O&M cost of this plant (1992) is $A 27,678, which is higher than Rattanasuk and Grinsukon's (1981) figure. The increase in Khon Kaen O&M costs from 1989-1992 was due to an increase in power and pumps repair costs. The O&M costs per capita, as shown in Table 4.12, ranged from $A 0.033 (1989) to $A 0.25 (1992).

Such O&M costs was found to be nearly the same as the O&M costs of WSPs in many countries as noted in Table 2.3. The comparison between the Khon Kaen WSP O&M costs to Reid's (1982) figure was documented in Table 2.2 or in Reid (1982) and Muiga (1982), Table 2.4, the costs computed including consumer price index increased shows the Khon Kaen WSP's O&M and those costs are nearly the same. The current Khon Kaen WSP O&M costs, as the City Council stated, were affordable to the City revenue.

Referring to Table 2.1, WSP was the most appropriate system for a city in terms of costs and affordability. It offered the least cost in both construction and operation compared to other systems. With low income and inability to access other systems, WSP was the best alternative for a city, especially in regional cities struggling with insufficient revenue. Another benefit to the public was that this plant could be supported by the lowest tax. Thus, in poor communities such as slums, the WSP seemed to be the better choice.

6.16.2 Plant personnel and operation problems

Currently, Khon Kaen WSP has four persons operating the plant. One is in charge of supervision and laboratory works. The other three are labourers. Referring to Mara (1982) in Table 3.5, Khon Kaen plant had six operators less. Compared to Arthur (1983), this plant was eleven persons lower in number.

From observation, most of the daytime inspection of the ponds was done by labourers on foot-trips. The hardest job for them was the removal of the floating algal mats. Such algal mat removal was found most difficult in ponds 3 to 5 in which there were no concrete slab linings and weeds were growing abundantly.

By using small handle nets and with only three persons working, the removal operation could not be coped with especially with algal mats present in every pond. The operators requested more labourers to help the team especially during the summer months. However, the City Council seemed unable to realise the necessity of the request (refer to discussion with operators). With the existing inability to remove algal mats efficiently, the mats were left settling into the bottom of the ponds.

The settling of mats occurred during early night time when pond temperature dropped. Upon observing a particular cluster of floating mats inside the pond, it was found that the mats at that point float up to the surface rapidly during the next day as temperature rose. Those mats became more milky in colour while producing a more offensive odour than before.

Whenever the mats were regularly removed, the ponds appeared to be of clear liquid, there were less mats formed and less odour detected. Further at this point, it could be observed that fish appeared in great numbers. During the study period, the operators dedicated nearly 85 percent of work-day time for algal mat removal. They also needed a small boat for operating this task, but no support was given at that time. At the east-west corners of the FPs, floating mats accumulated due to wind induction. It was only at this particular point that the operators could remove the mats. The other algal mat removal was done around the borders of the ponds which the operators could access. For MPs, such removal was done only near the outer boundary of the bank. The inadequate removal of algal mats in the MPs might also lead to the reduction of efficiency of the MPs performance.

Mowing the weeds was done once a month using a hand-mower. Cutting vegetation was found difficult in every pond due to steep embankments. Ponds 3 to 5 were also found to adversely affect mowing operation due to overgrowth of heavy weeds and the problem of trimming-walks on the irregular shapes of the bank. The operators also revealed that this situation was even more troublesome during the rainy season when heavy weeds proliferated and the pond banks were slippery.

Controlling the pumps is another time-consuming job since it has to be operated over a 24-hour period. Since the automatic control switch broke down in 1991, the pumps were from then on, operated manually. The switch board remained non-functional, as the City Councils viewed that a repair would be too costly.

Another difficulty in the plant was the pumps which often broke down (Picture 7). As observed, it was found that the grit chamber and the screen, which act as a preliminary screen for raw sewage prior to its discharge into the pump tank, were not functioning properly. Lots of detritus were found left inside these units. Plastics and other rigid materials were found inside the pump chamber thus making the pumps malfunction.

Picture 7 The broken pump system.

An investigation of the bar screen revealed that its mesh was quite large in scale, and as a result, allowed the plastics and rigid materials to pass through. This seemed to be one of the major causes of pump disorder. Heavy sludge in the pump tank might be the second factor causing pumps' operation disruption. After inspecting the pump tank, the sludge had greatly accumulated in this unit. The pumps were located underneath the liquid waste (submersible pumps) and were dipped into the sludge which could affect pump function.

As a result, the City Council considered that repair of the control switch would not help much, the redesign of these units would be a more appropriate alternative. However, to achieve this, the Council has to spend a huge amount on redesign work which they found to be unaffordable. The best alternative for them is to have the plant operated manually. This very high cost of pump repair would eventually increase the O&M costs.

Laboratory work was found to be done occasionally by the supervisor. He was in-charge, not only of the WSP plant, but also of handling public nuisances as well. Nuisances were not only generated from this plant but also from other sources. Thus, it seemed that this person worked in this plant on a part-time basis only. As a result, the laboratory work was limited. Only the bacterial tests were done fortnightly. The other parameters were seldom tested.

Lacking the necessary laboratory work for BOD, TSS, NO_3-N and PO_4-P, pH and temperature would make the Khon Kaen WSP incapable of providing information about the plant's efficiency in treating the city sewage. The overall performance of this plant was not known to the operators and the City Councils. The supervisor and the three operators were never trained about WSP system before. Their only information on WSP was from a handbook which briefly mentioned this plant. That handbook did not offer relevant information in solving problems currently encountered in the plant. All of the operators currently need training and recommendations from experienced WSP personnel.

Considering the usefulness of WSP to local workers, even with the presence of personnel problems, it could be mentioned that there are some benefits from this type of sewage treatment. These were (1) its encouragement to local workers and (2) and non-reliance on highly-skilled workers.

To a large extent, the operators revealed that this type of system was reliable and accessible for them even without much experience of it, as they could learn by observing the pond environment. Since they were local people, they were more familiar with the local atmosphere. To them, it seemed that the WSP was more nature-dependent and they could work with it by using local environment experiences. Furthermore, they felt that they could approach and learn to operate and solve the general problems generated by the plant.

Another point drawn from current skilled manpower inadequacy in the regional cities was that the Khon Kaen's WSP operation showed that even these unskilled workers could operate this plant efficiently. The problems encountered by operators were shortage of workers, and the defective plant design of preliminary units and pond structure. So, WSP seemed to be the most reliable for locals in terms of workforce opportunity.

CHAPTER 7

CONCLUSION AND RECOMMENDATIONS

7.1 The analysis of effluent discharged from the Khon Kaen WSP in comparison with Thailand's effluent standard (1989).

(1) The final effluent discharged from the Khon Kaen WSP had a BOD average of 14.8 mg/L which could meet the Thai standard (not exceeding 20 mg/L).

(2) The TSS from the final effluent with an average of 51.5 mg/L, did not meet the country's standard (not exceeding 30 mg/L).

(3) TDS in the final effluent of this plant, an average of 401 mg/L, could meet the standard (not exceeding 500 mg/L).

(4) The pH in the final effluent was an average of 8.1, meeting the standard (range of 5-9).

(5) Total coliform bacteria from the plant outflow, averaging 120.7 MPN/100 ml, could meet the standard (not exceeding 5,000 MPN/100 ml).

(6) Faecal coliform bacteria discharged from the plant, an average of 48.3 MPN/100 ml, could meet the standard (not exceeding 1,000 MPN/100 ml).

From (1) to (6), it could be summarised that the Khon Kaen WSP plant could efficiently treat Khon Kaen city sewage on the basis of the Thai standard, except for TSS which exceeded the standard value. The latter was attributed to the high algal content of the effluent.

7.2 The nitrogen and phosphorus levels discharged from the plant

These two groups of parameters even when not specified by the Thai standard will be adopted by the Khon Kaen WSP in the near future. Furthermore, as these two elements had significance in determining the cause of eutrophication in the receiving stream, this WSP could treat these two nutrients as follows:

(7) NH_3-N, NO_3-N and NO_2-N levels discharged from the plant had an average of 0.5, 0.2 and 0.3 mg/L respectively.

(8) PO_4-P level outflow from the plant, on average, was 0.0 mg/L.

Referring to (7) and (8), the Khon Kaen WSP could purify nutrients, N and P, at a level which might not cause eutrophication of the water body (the critical values of NO_3-N and

PO_4-P causing eutrophication, were 0.30 and 0.015 mg/L respectively (Stewart and Rohlich 1967).

Moreover, this WSP plant had efficient performance in reducing N and P up to the standard of advanced wastewater treatment in terms of NO_3-N (not exceeding 3.0 mg/L) and PO_4-P (not exceeding 1 mg/L).

7.3 Efficiency of Khon Kaen WSP plant

(9) The plant could reduce BOD by 80.1%, on average. Its high efficiency in BOD removal was mainly due to HRT, pond area and BOD loading.

(10) Overall efficiency in COD reduction of the plant was 33.9%. This lesser capacity in COD removal was as a result of high algal content.

(11) The plant could not remove TS, TDS and TSS. These three solids were increased in the final effluent compared to raw sewage with an average of 10.8%, 7.0% and 51.9% respectively.

(12) With nitrogen removal, the plant could reduce NH_3-N by 97.1%. This plant had no efficiency in reducing NO_3-N and NO_2-N as measured from the final effluent. Instead, these two parameters were increased by 7.8% and 650% respectively. Even if the Khon Kaen WSP has low efficiency in NO_3-N and NO_2-N removal, its final effluent contains very low levels of these two determinants as mentioned in (7). In this WSP, NH_3-N, being a large portion of nitrogenous compounds in this plant, was reduced primarily by algal uptake as nutrient.

(13) Phosphorus was removed 100.0% by the plant. This was due to algal culture utilising it as nutrient.

(14) The plant could reduce the total coliform by 99.9% and the faecal coliform by 99.9%.

The Khon Kaen plant had a very high efficiency in reducing BOD, PO_4-P, NH_3-N and total coliform and faecal coliform bacteria. The major functions in reducing these parameters were from bacteria oxidising BOD and algae culture uptaking nutrients.

7.4 Relationships between parameters

(15) The BOD loading and removal had a positive correlation in the facultative ponds. This simple linear equation, found in this investigation, might be applied to future facultative pond design in this region;

$$y = 0.6621x - 5.4941$$

where y = BOD removal (kg/ha d)
x = BOD loading (kg/ha d)

(16) The TS, TDS, TSS loading and removal were not strongly correlated and a conclusion could not be made since these relationships were more involved in complicated ecosystems.

(17) The NH_3-N, NO_3-N and NO_2-N loading-removal relationships were related depending on the degree of algae uptaking them as nutrients and nitrification reaction occurring inside the pond.

(18) The PO_4-P loading and removal was positively related in every pond, and the positive correlation pattern was more than in solid and nitrogen models.

A correlation between loading and removal of BOD and PO_4-P would be advantageous for future development of WSP in Thailand. For solids and nitrogen, the correlation was not clear and was inconclusive.

(19) The BOD level was negatively related to HRT.

(20) The BOD and COD levels were related only in the first pond which had the highest values of BOD, COD, solids, NH_3-N, and PO_4-P. For the other ponds which had low levels of these determinants, no correlation was found.

(21) The TSS was positively correlated to BOD, COD and DO.

(22) The TSS was negatively correlated to NH_3-N.

(23) The pH had a positive correlation with DO, NH_3-N and TSS.

(24) The pH was negatively correlated to total coliform and faecal coliform numbers.

(25) A negative correlation existed between temperature and BOD level.

(26) Temperature had a positive correlation with percent BOD removal.

The BOD was the most important determinant and according to this study, to reduce BOD levels in WSP, increasing HRT would help since it increases BOD removal. Temperature was found to be another important factor in reducing BOD; a higher BOD removal rate was dependent on temperature increase. Majority of nitrogenous compound, in the form NH_3-N, was reduced by algal culture uptake. The degree of NH_3-N uptake by algae was dependent on photosynthesis.

7.5 The daytime behaviour of some wastewater determinants in Khon Kaen WSP.

(27) Light intensity was at its highest level at 12 mid-day.

(28) The pH and DO were at a high peak during the period, 1-2 pm.

(29) The PO_4-P and NH_3-N were at high levels in the early morning which tended to drop during late afternoon.

(30) The NO_3-N was at a high peak at noon which dropped during late afternoon.

(31) The light intensity was positively related to pH and DO.

(32) The pH was positively correlated to DO and temperature.

From (27) to (32), it could be concluded that whenever light intensity was at a high level, active photosynthesis would be encouraged to occur in the ponds which finally led to increased pH and DO levels. At the same time, PO_4-P and NH_3-N were removed as algal nutrient. Another portion of NH_3-N was also reduced in level by nitrification to NO_3-N which attained higher values at noon.

7.6 The three main green algal species found inhabiting this plant were, *Chlamydomonas* spp., *Pandorina* spp. and *Scenedesmus* spp. *Oscillatoria* spp., *Spirulina* spp., and *Anacystis* spp. were the major blue-green algal species existing in the ponds. *Anacystis* spp. was the dominant algal species functioning in the Khon Kaen WSP. Heavy algal mats and offensive odour were mainly produced by this species.

7.7 *Oreochromis niloticus* and *Puntius gonionatus* were the main fish species in the Khon Kaen WSP which were found to help in the TSS removal as well as being the protein source of the people who live within the pond vicinity.

7.8 The plant needed at least 3 more operators to handle the pond operation. The present operators required more training, especially in areas associated with the WSP process. Essential wastewater parameter tests which reflect pond performance were required.

7.9 The structure of some ponds was inappropriate in terms of shape, area, embankment lining and slope. Those were the factors found reducing the pond performance in sewage purification, causing bank erosion and enhancing vegetation growth.

7.10 The preliminary treatment process in this plant caused the pumps to malfunction. The pumps were often broken down by incoming plastics and rigid materials.

7.11 Some conventional WSP design equations were inappropriate, especially the equations involved in BOD loading-removal. Recommended BOD loading by some researchers could be found applicable to this plant. The most appropriate means for future WSP design method was to construct the BOD loading-removal equation experienced in a neighbouring pond, as in (15), or from the pilot-scale.

7.12 This plant could show the cheapest sewage treatment in both construction and operation costs compared to other systems without compromising the quality of effluent. It was a cost-effective sewage treatment plant.

7.13 Public nuisance against the WSP plant was caused by heavy algal mats. The operators could not completely remove the mats from the ponds. Some ponds were nearly clogged with these mats and the liquid movement between these ponds was found to be difficult. The algal mats, in addition, was found to reduce the efficiency of pond performance.

7.14 According to the country's policy of wastewater management, this WSP was one of the best alternatives in terms of cost-effectiveness. The WSP performance in this study justified its role in the regional city's socio-economic status.

7.15 Recommendations

(1) Effective methods should be initiated in reducing TSS level. This could be done by conducting future studies such as (1) fish culture experiments (2) investigations involving slow sand filtration and chemicals.

(2) Full-scale research on the capacity of WSP in BOD and other major pollutants in the form of loading-removal relationships associated with the local environment should be done by collecting data throughout the year. These data should be associated with variation by seasons and locations of the WSPs in order to achieve an average value of a relationship, as exemplified in (15).

(3) The efficiency of the ponds could be improved by appropriate measures such as adjusting the shape, lining the embankment, increasing HRT and extending the ponds' area. These should be areas of future research regarding WSP development.

(4) Studies about toxins released by algae in association with fish and human health were also necessary since pond fishes were used for human consumption.

(5) To understand the functions and behaviour of the dominant algal species in the ponds, varying temperature and seasons should be studied. This is because algae was the most important bioactor functioning inside the ponds and symbiotically related to bacteria. The search for effective

measures in removing algal mats should be another area of concern which would reduce public nuisance and increase efficiency of the ponds.

(6) Improving preliminary units was essential in order to increase the pumps' efficiency. This would lead to the reduction of the operation and maintenance costs on a long-term basis.

(7) Initiating laboratory works associated with essential parameters required for testing the WSP's efficiency should be a foremost priority.

(8) Training plant personnel about WSP process is also necessary since this would help them understand the WSP operation comprehensively. Thus, it would result in an increase of the pond's efficiency and problems arising from the WSP would also be minimised.

APPENDIX 1. Detailed performance data of Khon Kaen WSP.

Date	Sewage flow(cu.m)	Pond volume	(cu.m)	Pond area	(ha)	HRT (d)	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5	Total
24/1/93	7410	Pond 1	264200	Pond 1	13.2		36	12	3	1	2	53
28/1/93	7829	Pond 2	90000	Pond 2	4.5		34	11	3	1	2	51
1/2/93	8374	Pond 3	19800	Pond 3	0.99		32	11	2	1	2	47
5/2/93	8130	Pond 4	7200	Pond 4	0.36		32	11	2	1	2	49
9/2/93	7560	Pond 5	14400	Pond 5	0.72		35	12	3	1	2	52
13/2/93	8294	Total	395600	Total	19.8		32	11	2	1	2	48
17/2/93	9340						28	10	2	1	2	42
21/2/93	8541						31	11	2	1	2	46
25/2/93	8380						32	11	2	1	2	47
1/3/93	8127						33	11	2	1	2	49
5/3/93	7949						33	11	2	1	2	50
9/3/93	7751						34	12	3	1	2	51
13/3/93	7309						36	12	3	1	2	54
17/3/93	6974						38	13	3	1	2	57
21/3/93	7511						35	12	3	1	2	53
25/3/93	7039						38	13	3	1	2	56
29/3/93	7588						35	12	3	1	2	52
2/4/93	8142						32	11	2	1	2	49
6/4/93	8391						31	11	2	1	2	47
10/4/93	7902						33	11	3	1	2	50
14/4/93	7575						35	12	3	1	2	52
18/4/93	7894						33	11	3	1	2	50
22/4/93	7911						33	11	3	1	2	50
26/4/93	8021						33	11	2	1	2	49
Average	7914.3						33.5	11.4	2.5	0.9	1.8	50

APPENDIX 1 (cont.)

Date	BOD loading	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5	Areal BOD loading	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5
24/1/93	(kg/d)	432.0	224.5	160.8	143.0	133.4	(kg BOD/ha-d)	32.7	49.9	162.4	397.3	185.3
28/1/93		409.5	209.0	185.5	127.6	117.4		31.0	46.5	187.4	354.5	163.1
1/2/93		365.9	211.9	178.4	165.0	159.1		27.7	47.1	180.2	458.2	221.0
5/2/93		449.6	248.8	208.9	165.0	144.7		34.1	55.3	211.1	458.4	201.0
9/2/93		511.8	224.5	164.1	140.6	126.3		38.8	49.9	165.7	390.6	175.4
13/2/93		549.9	229.7	196.6	165.9	160.1		41.7	51.1	198.6	460.8	222.3
17/2/93		579.1	318.5	286.7	255.0	242.8		43.9	70.8	289.6	708.3	337.3
21/2/93		668.8	344.2	236.6	185.3	153.7		50.7	76.5	239.0	514.8	213.5
25/2/93		687.2	337.7	270.7	212.0	190.2		52.1	75.0	273.4	588.9	264.2
1/3/93		650.2	307.2	221.9	200.7	170.7		49.3	68.3	224.1	557.6	237.0
5/3/93		612.1	302.1	220.2	185.2	148.6		46.4	67.1	222.4	514.5	206.5
9/3/93		527.1	239.5	211.6	162.8	129.4		39.9	53.2	213.7	452.1	179.8
13/3/93		615.4	245.6	165.9	126.4	111.8		46.6	54.6	167.6	351.2	155.3
17/3/93		638.8	266.4	134.6	109.5	90.7		48.4	59.2	136.0	304.1	125.9
21/3/93		655.7	220.1	110.4	87.9	75.1		49.7	48.9	111.5	244.1	104.3
25/3/93		727.8	317.5	142.9	128.8	117.6		55.1	70.5	144.3	357.8	163.3
29/3/93		703.4	321.0	164.7	126.7	119.1		53.3	71.3	166.3	352.0	165.5
2/4/93		641.6	274.4	144.1	132.7	124.6		48.6	61.0	145.6	368.7	173.0
6/4/93		680.5	329.8	198.9	176.2	170.3		51.6	73.3	200.9	489.5	236.6
10/4/93		529.4	239.4	187.3	160.4	155.7		40.1	53.2	189.2	445.6	216.2
14/4/93		449.2	217.4	149.2	131.0	115.9		34.0	48.3	150.7	364.0	161.0
18/4/93		539.2	255.0	142.1	126.3	120.8		40.8	56.7	143.5	350.8	167.7
22/4/93		663.7	269.0	176.4	144.8	132.1		50.3	59.8	178.2	402.1	183.5
26/4/93		747.6	328.9	194.9	162.8	154.8		56.6	73.1	196.9	452.3	215.0
Average		584.8	270.1	185.6	155.1	140.2		44.3	60.0	187.4	430.8	194.7

APPENDIX 1 (cont.)

Date	BOD	Raw sewage	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5	Areal BOD	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5
24/1/93	(mg/L)	58.3	30.3	21.7	19.3	18.0	15.3	removal	15.7	14.2	18.0	26.8	27.8
28/1/93		52.3	26.7	23.7	16.3	15.0	12.3	(kg BOD/ha d	15.2	5.2	58.5	28.3	29.4
1/2/93		43.7	25.3	21.3	19.7	19.0	17.0		11.7	7.4	13.5	16.3	23.3
5/2/93		55.3	30.6	25.7	20.3	17.8	15.3		15.2	8.9	44.3	56.5	28.2
9/2/93		67.7	29.7	21.7	18.6	16.7	16.7		21.8	13.4	23.7	39.9	0.0
13/2/93		66.3	27.7	23.7	20.0	19.3	17.7		24.3	7.4	31.0	16.1	18.4
17/2/93		62.0	34.1	30.7	27.3	26.0	21.7		19.7	7.1	32.1	33.7	55.8
21/2/93		78.3	40.3	27.7	21.7	18.0	16.3		24.6	23.9	51.8	87.8	20.2
25/2/93		82.0	40.3	32.3	25.3	22.7	17.7		26.5	14.9	59.3	60.5	58.2
1/3/93		80.0	37.8	27.3	24.7	21.0	16.0		26.0	19.0	21.3	83.5	56.4
5/3/93		77.0	38.0	27.7	23.3	18.7	14.7		23.5	18.2	35.3	101.6	44.2
9/3/93		68.0	30.9	27.3	21.0	16.7	12.7		21.8	6.2	49.3	92.6	43.1
13/3/93		84.2	33.6	22.7	17.3	15.3	10.3		28.0	17.7	39.9	40.6	50.8
17/3/93		91.6	38.2	19.3	15.7	13.0	8.7		28.2	29.3	25.4	52.3	41.7
21/3/93		87.3	29.3	14.7	11.7	10.0	6.3		33.0	24.4	22.8	35.5	38.6
25/3/93		103.4	45.1	20.3	18.3	16.7	12.7		31.1	38.8	14.2	31.3	39.1
29/3/93		92.7	42.3	21.7	16.7	15.7	14.3		29.0	34.7	38.3	21.1	14.8
2/4/93		78.8	33.7	17.7	16.3	15.3	13.3		27.8	28.9	11.5	22.6	22.6
6/4/93		81.1	39.3	23.7	21.0	20.3	18.0		26.6	29.1	22.9	16.3	26.8
10/4/93		67.0	30.3	23.7	20.3	19.7	17.7		22.0	11.6	27.1	13.2	22.0
14/4/93		59.3	28.7	19.7	17.3	15.3	14.0		17.6	15.2	18.4	42.1	13.7
18/4/93		68.3	32.3	18.0	16.0	15.3	12.7		21.5	25.1	15.9	15.3	28.5
22/4/93		83.9	34.0	22.3	18.3	16.7	15.7		29.9	20.6	32.0	35.2	11.0
26/4/93		93.2	41.0	24.3	20.3	19.3	18.0		31.7	29.8	32.4	22.3	14.5
Average		74.2	34.1	23.3	19.4	17.6	14.8		23.8	18.8	30.8	41.3	30.4

APPENDIX 1 (cont.)

Date	BOD removal	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5	Total	COD	Raw sewage	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5
24/1/93	(%)	48.0	28.4	11.1	6.7	15.0	73.8	(mg/L)	121	243	188	135	120	94
28/1/93		48.9	11.2	31.2	8.0	18.0	76.5		112	211	180	126	111	86
1/2/93		42.1	15.8	7.5	3.6	10.5	61.1		103	199	167	122	109	84
5/2/93		44.7	16.0	21.0	12.3	14.0	72.3		110	205	147	130	114	90
9/2/93		56.1	26.9	14.3	10.2	0.0	75.3		139	257	196	144	128	101
13/2/93		58.2	14.4	15.6	3.5	8.3	73.3		117	261	188	130	113	91
17/2/93		45.0	10.0	11.1	4.8	16.5	65.0		100	286	175	124	100	82
21/2/93		48.5	31.3	21.7	17.1	9.4	79.2		103	291	169	125	105	87
25/2/93		50.9	19.9	21.7	10.3	22.0	78.4		118	282	178	136	111	92
1/3/93		52.8	27.8	9.5	15.0	23.8	80.0		130	244	193	155	123	97
5/3/93		50.6	27.1	15.9	19.7	21.4	80.9		116	200	179	136	112	87
9/3/93		54.6	11.7	23.1	20.5	24.0	81.3		109	191	135	122	108	70
13/3/93		60.1	32.4	23.8	11.6	32.7	87.8		139	182	160	145	119	70
17/3/93		58.3	49.5	18.7	17.2	33.1	90.5		146	228	177	139	122	84
21/3/93		66.4	49.8	20.4	14.5	37.0	92.8		149	237	186	129	117	81
25/3/93		56.4	55.0	9.9	8.7	24.0	87.7		179	269	185	131	125	81
29/3/93		54.4	48.7	23.0	6.0	8.9	84.6		155	254	179	129	116	85
2/4/93		57.2	47.5	7.9	6.1	13.1	83.1		128	231	184	137	114	75
6/4/93		51.5	39.7	11.4	3.3	11.3	77.8		130	244	197	148	120	82
10/4/93		54.8	21.8	14.3	3.0	10.2	73.6		112	231	152	129	118	67
14/4/93		51.6	31.4	12.2	11.6	8.5	76.4		102	198	149	119	104	62
18/4/93		52.7	44.3	11.1	4.4	17.0	81.4		115	184	158	120	100	71
22/4/93		59.5	34.4	17.9	8.7	6.0	81.3		130	196	169	122	115	81
26/4/93		56.0	40.7	16.5	4.9	6.7	80.7		148	184	152	131	124	93
Average		53.3	30.7	16.3	9.7	16.3	78.9		125.5	229.5	172.6	131.8	114.5	83.0

APPENDIX 1 (cont.)

Date	COD removal (%)	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5	Total	TS (mg/L)	Raw sewage	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5
24/1/93		-100	23	28	11	22	23		385	671	500	499	487	483
28/1/93		-88	15	30	12	23	23		353	632	481	450	421	391
1/2/93		-93	16	27	11	23	19		347	622	476	442	431	377
5/2/93		-86	28	12	12	21	18		487	673	631	618	595	571
9/2/93		-84	24	27	11	21	28		508	673	612	589	574	511
13/2/93		-123	28	31	13	19	22		463	653	580	577	501	496
17/2/93		-187	39	29	19	18	18		421	661	522	500	484	475
21/2/93		-183	42	26	16	17	15		471	682	511	489	484	482
25/2/93		-139	37	24	18	17	22		464	679	505	492	479	471
1/3/93		-88	21	20	21	21	25		394	601	499	471	463	459
5/3/93		-72	11	24	18	22	25		383	622	485	478	459	441
9/3/93		-75	29	10	11	35	36		366	568	477	463	444	437
13/3/93		-31	12	9	18	41	50		428	622	500	485	469	451
17/3/93		-56	22	21	12	31	43		444	693	489	471	459	434
21/3/93		-59	22	31	9	31	46		394	657	499	486	474	456
25/3/93		-51	31	29	5	35	55		364	673	485	463	455	450
29/3/93		-64	30	28	10	27	45		381	651	509	474	461	429
2/4/93		-81	20	26	17	34	41		415	634	511	496	477	451
6/4/93		-87	19	25	19	32	37		392	624	585	493	487	452
10/4/93		-107	34	15	9	43	40		386	600	566	521	481	468
14/4/93		-94	25	20	13	40	39		359	599	432	421	400	396
18/4/93		-60	14	24	17	29	38		354	612	527	398	382	376
22/4/93		-51	14	28	6	30	38		403	618	467	453	449	437
26/4/93		-24	17	14	5	25	37		442	622	490	487	472	465
Average		-86.8	23.8	23.2	13.0	27.4	32.6		408.5	639.3	514.1	488.2	470.3	452.5

APPENDIX 1 (cont.)

Date	TS loading	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5	Areal TS loading	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5
24/1/93	(kg/d)	2852.9	4972.1	3705.0	3697.6	3608.7	(kg/ha d)	216.1	1104.9	3742.4	10271.1	5012.0
28/1/93		2763.6	4947.9	3765.7	3523.1	3296.0		209.4	1099.5	3803.8	9786.3	4577.8
1/2/93		2905.8	5208.6	3986.0	3701.3	3609.2		220.1	1157.5	4026.3	10281.4	5012.8
5/2/93		3959.3	5471.5	5130.0	5024.3	4837.4		299.9	1215.9	5181.8	13956.5	6718.5
9/2/93		3840.5	5087.9	4626.7	4452.8	4339.4		290.9	1130.6	4673.5	12369.0	6027.0
13/2/93		3840.1	5416.0	4810.5	4785.6	4155.3		290.9	1203.6	4859.1	13293.4	5771.2
17/2/93		3932.1	6173.7	4875.5	4670.0	4520.6		297.9	1371.9	4924.7	12972.2	6278.6
21/2/93		4022.8	5825.0	4364.5	4176.5	4133.8		304.8	1294.4	4408.5	11601.5	5741.5
25/2/93		3888.3	5690.0	4231.9	4123.0	4014.0		294.6	1264.4	4274.6	11452.7	5575.0
1/3/93		3202.0	4884.3	4055.4	3827.8	3762.8		242.6	1085.4	4096.3	10632.8	5226.1
5/3/93		3044.5	4944.3	3855.3	3799.6	3648.6		230.6	1098.7	3894.2	10554.5	5067.5
9/3/93		2836.9	4402.6	3697.2	3588.7	3441.4		214.9	978.3	3734.6	9968.6	4779.8
13/3/93		3128.3	4546.2	3654.5	3544.9	3427.9		237.0	1010.3	3691.4	9846.8	4761.0
17/3/93		3096.5	4833.0	3410.3	3284.8	3201.1		234.6	1074.0	3444.7	9124.3	4445.9
21/3/93		2959.3	4934.7	3748.0	3650.3	3560.2		224.2	1096.6	3785.8	10139.9	4944.7
25/3/93		2562.2	4737.2	3413.9	3259.1	3202.7		194.1	1052.7	3448.4	9052.9	4448.3
29/3/93		2891.0	4939.8	3862.3	3596.7	3498.1		219.0	1097.7	3901.3	9990.9	4858.4
2/4/93		3378.9	5162.0	4160.6	4038.4	3883.7		256.0	1147.1	4202.6	11217.9	5394.1
6/4/93		3289.3	5236.0	4908.7	4136.8	4086.4		249.2	1163.6	4958.3	11491.0	5675.6
10/4/93		3050.2	4741.2	4472.5	4116.9	3800.9		231.1	1053.6	4517.7	11436.0	5279.0
14/4/93		2719.4	4537.4	3272.4	3189.1	3030.0		206.0	1008.3	3305.5	8858.5	4208.3
18/4/93		2794.5	4831.1	4160.1	3141.8	3015.5		211.7	1073.6	4202.2	8727.3	4188.2
22/4/93		3188.1	4889.0	3694.4	3583.7	3552.0		241.5	1086.4	3731.8	9954.7	4933.4
26/4/93		3545.3	4989.1	3930.3	3906.2	3785.9		268.6	1108.7	3970.0	10850.6	5258.2
Average		3237.2	5058.4	4074.7	3867.5	3725.5		245.2	1124.1	4115.8	10743.0	5174.3

APPENDIX 1 (cont.)

Date	TS removal	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5	TS removal	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5	Total
24/1/93	(kg/ha d)	-160.6	281.6	7.5	247.0	41.2	(%)	-74.3	25.5	0.2	2.4	0.8	-25.5
28/1/93		-165.5	262.7	245.2	630.7	326.2		-79.0	23.9	6.4	6.4	7.1	-10.8
1/2/93		-174.5	271.7	287.6	255.9	628.1		-79.3	23.5	7.1	2.5	12.5	-8.6
5/2/93		-114.6	75.9	106.8	519.4	271.0		-38.2	6.2	2.1	3.7	4.0	-17.2
9/2/93		-94.5	102.5	175.6	315.0	661.5		-32.5	9.1	3.8	2.5	11.0	-0.6
13/2/93		-119.4	134.5	25.1	1751.0	57.6		-41.0	11.2	0.5	13.2	1.0	-7.1
17/2/93		-169.8	288.5	207.6	415.1	116.8		-57.0	21.0	4.2	3.2	1.9	-12.8
21/2/93		-136.5	324.6	189.8	118.6	23.7		-44.8	25.1	4.3	1.0	0.4	-2.3
25/2/93		-136.5	324.0	110.0	302.6	93.1		-46.3	25.6	2.6	2.6	1.7	-1.5
1/3/93		-127.4	184.2	229.9	180.6	45.2		-52.5	17.0	5.6	1.7	0.9	-16.5
5/3/93		-143.9	242.0	56.2	419.5	198.7		-62.4	22.0	1.4	4.0	3.9	-15.1
9/3/93		-118.6	156.7	109.6	409.1	75.4		-55.2	16.0	2.9	4.1	1.6	-19.4
13/3/93		-107.4	198.2	110.7	324.8	182.7		-45.3	19.6	3.0	3.3	3.8	-5.4
17/3/93		-131.6	316.2	126.8	232.5	242.2		-56.1	29.4	3.7	2.5	5.4	2.3
21/3/93		-149.7	263.7	98.6	250.4	187.8		-66.8	24.0	2.6	2.5	3.8	-15.7
25/3/93		-164.8	294.1	156.4	156.4	48.9		-84.9	27.9	4.5	1.7	1.1	-23.6
29/3/93		-155.2	239.4	268.3	274.0	337.2		-70.9	21.8	6.9	2.7	6.9	-12.6
2/4/93		-135.1	222.5	123.4	429.7	294.0		-52.8	19.4	2.9	3.8	5.5	-8.7
6/4/93		-147.5	72.7	779.8	139.9	407.9		-59.2	6.3	15.7	1.2	7.2	-15.3
10/4/93		-128.1	59.7	359.2	878.0	142.7		-55.4	5.7	8.0	7.7	2.7	-21.2
14/4/93		-137.7	281.1	84.2	441.9	42.1		-66.9	27.9	2.5	5.0	1.0	-10.3
18/4/93		-154.3	149.1	1028.6	350.8	65.8		-72.9	13.9	24.5	4.0	1.6	-6.2
22/4/93		-128.9	265.5	111.9	87.9	131.9		-53.3	24.4	3.0	0.9	2.7	-8.4
26/4/93		-109.4	235.3	24.3	334.2	78.0		-40.7	21.2	0.6	3.1	1.5	-5.2
Average		-138.0	218.6	209.3	394.4	195.8		-57.8	19.5	5.0	3.6	3.7	-11.2

APPENDIX 1 (cont.)

Date	TSS	Raw sewage	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5	TSS loading	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5
24/1/93	(mg/L)	33	125	89	81	72	67	(kg/d)	244.5	926.3	659.5	600.2	533.5
28/1/93		28	101	71	66	63	35		219.2	790.7	555.9	516.7	493.2
1/2/93		14	95	67	63	59	29		117.2	795.5	561.1	527.6	494.1
5/2/93		39	126	131	97	81	76		317.1	1024.4	1065.0	788.6	658.5
9/2/93		49	128	127	88	79	71		370.4	967.7	960.1	665.3	597.2
13/2/93		37	122	114	85	75	69		306.9	1011.9	945.5	705.0	622.1
17/2/93		35	138	96	79	72	61		326.9	1288.9	896.6	737.9	672.5
21/2/93		47	141	93	75	70	68		401.4	1204.3	794.3	640.6	597.9
25/2/93		43	139	90	79	67	64		360.3	1164.8	754.2	662.0	561.5
1/3/93		40	105	87	71	62	62		325.1	853.3	707.0	577.0	503.9
5/3/93		21	114	85	74	59	43		166.9	906.2	675.7	588.2	469.0
9/3/93		17	107	81	64	47	41		131.8	829.4	627.8	496.1	364.3
13/3/93		31	116	87	76	61	49		226.6	847.8	635.9	555.5	445.8
17/3/93		39	123	83	69	57	41		272.0	857.8	578.8	481.2	397.5
21/3/93		37	118	87	84	64	51		277.9	886.3	653.5	630.9	480.7
25/3/93		28	135	83	66	53	49		197.1	950.3	584.2	464.6	373.1
29/3/93		32	123	86	71	58	39		242.8	933.3	652.6	538.7	440.1
2/4/93		41	119	96	75	64	48		333.8	968.9	781.6	610.7	521.1
6/4/93		35	112	117	70	73	49		293.7	939.8	981.7	587.4	612.5
10/4/93		31	105	94	82	70	57		245.0	829.7	742.8	648.0	553.1
14/4/93		25	100	73	62	48	37		189.4	757.5	553.0	469.7	363.6
18/4/93		22	124	95	49	46	30		173.7	978.9	749.9	386.8	363.1
22/4/93		40	125	84	67	51	42		316.4	988.9	664.5	530.0	403.5
26/4/93		49	138	87	77	59	57		393.0	1106.9	697.8	617.6	473.2
Average		33.9	120.0	91.8	73.8	62.9	51.5		268.7	950.4	728.3	584.4	499.8

APPENDIX 1 (cont.)

Date	TSS loading	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5	TSS removal	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5
24/1/93	(kg/ha d)	18.5	70.2	50.0	45.5	40.4	(kg/ha d)	-51.6	59.3	59.9	185.3	51.5
28/1/93		16.6	59.9	42.1	39.1	37.4		-43.3	52.2	39.5	65.2	304.5
1/2/93		8.9	60.3	42.5	40.0	37.4		-51.4	52.1	33.8	93.0	348.9
5/2/93		24.0	77.6	80.7	59.7	49.9		-53.6	-9.0	279.2	361.3	56.5
9/2/93		28.1	73.3	72.7	50.4	45.2		-45.2	1.7	297.8	189.0	84.0
13/2/93		23.2	76.7	71.6	53.4	47.1		-53.4	14.7	243.0	230.4	69.1
17/2/93		24.8	97.6	67.9	55.9	50.9		-72.9	87.2	160.4	181.6	142.7
21/2/93		30.4	91.2	60.2	48.5	45.3		-60.8	91.1	155.3	118.6	23.7
25/2/93		27.3	88.2	57.1	50.2	42.5		-60.9	91.2	93.1	279.3	34.9
1/3/93		24.6	64.6	53.6	43.7	38.2		-40.0	32.5	131.3	203.2	0.0
5/3/93		12.6	68.7	51.2	44.6	35.5		-56.0	51.2	88.3	331.2	176.6
9/3/93		10.0	62.8	47.6	37.6	27.6		-52.8	44.8	133.1	366.0	64.6
13/3/93		17.2	64.2	48.2	42.1	33.8		-47.1	47.1	81.2	304.5	121.8
17/3/93		20.6	65.0	43.9	36.5	30.1		-44.4	62.0	98.6	232.5	155.0
21/3/93		21.1	67.1	49.5	47.8	36.4		-46.1	51.7	22.8	417.3	135.6
25/3/93		14.9	72.0	44.3	35.2	28.3		-57.1	81.3	120.9	254.2	39.1
29/3/93		18.4	70.7	49.4	40.8	33.3		-52.3	62.4	115.0	274.0	200.2
2/4/93		25.3	73.4	59.2	46.3	39.5		-48.1	41.6	172.7	248.8	180.9
6/4/93		22.2	71.2	74.4	44.5	46.4		-48.9	-9.3	398.4	-69.9	279.7
10/4/93		18.6	62.9	56.3	49.1	41.9		-44.3	19.3	95.8	263.4	142.7
14/4/93		14.3	57.4	41.9	35.6	27.5		-43.0	45.5	84.2	294.6	115.7
18/4/93		13.2	74.2	56.8	29.3	27.5		-61.0	50.9	366.8	65.8	175.4
22/4/93		24.0	74.9	50.3	40.2	30.6		-50.9	72.1	135.8	351.6	98.9
26/4/93		29.8	83.9	52.9	46.8	35.9		-54.1	90.9	81.0	401.1	22.3
Average		20.4	72.0	55.2	44.3	37.9		-51.6	49.4	145.3	235.1	126.0

APPENDIX 1 (cont.)

Date	TSS removal	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5	Total	TDS	Raw sewage	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5
24/1/93	(%)	-278.8	28.8	9.0	11.1	6.9	-103.0	(mg/L)	352	546	411	418	415	416
28/1/93		-260.7	29.7	7.0	4.5	44.4	-25.0		325	531	410	384	358	356
1/2/93		-578.6	29.5	6.0	6.3	50.8	-107.1		333	527	409	379	372	348
5/2/93		-223.1	-4.0	26.0	16.5	6.2	-94.9		448	547	500	521	514	495
9/2/93		-161.2	0.8	30.7	10.2	10.1	-44.9		459	545	485	501	495	440
13/2/93		-229.7	6.6	25.4	11.8	8.0	-86.5		426	531	466	492	426	427
17/2/93		-294.3	30.4	17.7	8.9	15.3	-74.3		386	523	426	421	412	414
21/2/93		-200.0	34.0	19.4	6.7	2.9	-44.7		424	541	418	414	414	414
25/2/93		-223.3	35.3	12.2	15.2	4.5	-48.8		421	540	415	413	412	407
1/3/93		-162.5	17.1	18.4	12.7	0.0	-55.0		354	496	412	400	401	397
5/3/93		-442.9	25.4	12.9	20.3	27.1	-104.8		362	508	400	404	400	398
9/3/93		-529.4	24.3	21.0	26.6	12.8	-141.2		349	461	396	399	397	396
13/3/93		-274.2	25.0	12.6	19.7	19.7	-58.1		397	506	413	409	408	402
17/3/93		-215.4	32.5	16.9	17.4	28.1	-5.1		405	570	406	402	402	393
21/3/93		-218.9	26.3	3.4	23.8	20.3	-37.8		357	539	412	402	410	405
25/3/93		-382.1	38.5	20.5	19.7	7.5	-75.0		336	538	402	397	402	401
29/3/93		-284.4	30.1	17.4	18.3	32.8	-21.9		349	528	423	403	403	390
2/4/93		-190.2	19.3	21.9	14.7	25.0	-17.1		374	515	415	421	413	403
6/4/93		-220.0	-4.5	40.2	-4.3	32.9	-40.0		357	512	468	423	414	403
10/4/93		-238.7	10.5	12.8	14.6	18.6	-83.9		355	495	472	439	411	411
14/4/93		-300.0	27.0	15.1	22.6	22.9	-48.0		334	499	359	359	352	359
18/4/93		-463.6	23.4	48.4	6.1	34.8	-36.4		332	488	432	349	336	346
22/4/93		-212.5	32.8	20.2	23.9	17.6	-5.0		363	493	383	386	398	395
26/4/93		-181.6	37.0	11.5	23.4	3.4	-16.3		393	484	403	410	413	408
Average		-281.9	23.2	18.6	14.7	18.9	-57.3		374.6	519.3	422.3	414.4	407.4	401.0

APPENDIX 1 (cont.)

Date	TDS loading (kg/d)	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5	Areal TDS loading (kg/ha d)	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5
24/1/93		2608.3	4045.9	3045.5	3097.4	3075.2		197.6	899.1	3076.3	8603.8	4271.0
28/1/93		2544.4	4157.2	3209.9	3006.3	2652.8		192.8	923.8	3242.3	8350.9	3684.4
1/2/93		2788.5	4413.1	3425.0	3173.7	2756.5		211.3	980.7	3459.6	8816.0	3828.5
5/2/93		3642.2	4447.1	4065.0	4235.7	3808.7		275.9	988.2	4106.1	11765.9	5289.9
9/2/93		3470.0	4120.2	3666.6	3787.6	3668.0		262.9	915.6	3703.6	10521.0	5094.4
13/2/93		3533.2	4404.1	3865.0	4080.6	3156.7		267.7	978.7	3904.0	11335.1	4384.3
17/2/93		3605.2	4884.8	3978.8	3932.1	3052.9		273.1	1085.5	4019.0	10922.6	4240.2
21/2/93		3621.4	4620.7	3570.1	3536.0	3067.7		274.3	1026.8	3606.2	9822.2	4260.8
25/2/93		3528.0	4525.2	3477.7	3460.9	3052.9		267.3	1005.6	3512.8	9613.7	4240.2
1/3/93		2877.0	4031.0	3348.3	3250.8	2971.4		218.0	895.8	3382.1	9030.0	4127.0
5/3/93		2877.5	4038.1	3179.6	3211.4	2964.0		218.0	897.4	3211.7	8920.5	4116.7
9/3/93		2705.1	3573.2	3069.4	3092.6	2941.8		204.9	794.0	3100.4	8590.7	4085.8
13/3/93		2901.7	3698.4	3018.6	2989.4	3023.3		219.8	821.9	3049.1	8303.8	4199.0
17/3/93		2824.5	3975.2	2831.4	2803.5	2978.8		214.0	883.4	2860.0	7787.6	4137.3
21/3/93		2681.4	4048.4	3094.5	3019.4	3038.1		203.1	899.7	3125.8	8387.3	4219.6
25/3/93		2365.1	3787.0	2829.7	2794.5	2978.8		179.2	841.6	2858.3	7762.5	4137.3
29/3/93		2648.2	4006.5	3209.7	3058.0	2986.2		200.6	890.3	3242.1	8494.3	4147.5
2/4/93		3045.1	4193.1	3378.9	3427.8	3060.3		230.7	931.8	3413.1	9521.6	4250.5
6/4/93		2995.6	4296.2	3927.0	3549.4	3067.7		226.9	954.7	3966.7	9859.4	4260.8
10/4/93		2805.2	3911.5	3729.7	3469.0	3045.5		212.5	869.2	3767.4	9636.1	4229.9
14/4/93		2530.1	3779.9	2719.4	2719.4	2608.3		191.7	840.0	2746.9	7554.0	3622.7
18/4/93		2620.8	3852.3	3410.2	2755.0	2489.8		198.5	856.1	3444.7	7652.8	3458.0
22/4/93		2871.7	3900.1	3029.9	3053.6	2949.2		217.6	866.7	3060.5	8482.4	4096.1
26/4/93		3152.3	3882.2	3232.5	3288.6	3060.3		238.8	862.7	3265.1	9135.0	4250.5
Average		2968.4	4108.0	3346.4	3283.0	3019.0		224.9	912.9	3380.2	9119.6	4193.0

APPENDIX 1 (cont.)

Date	TDS removal	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5	TDS removal	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5	Total
24/1/93	(kg/ha d)	-108.9	222.3	-52.4	61.8	-10.3	(%)	-35.5	32.8	-1.7	0.7	-0.2	-18.2
28/1/93		-122.2	210.5	205.6	565.4	21.7		-38.8	29.5	6.8	7.3	0.6	-9.5
1/2/93		-123.1	219.6	253.8	162.8	279.1		-36.8	28.9	7.9	1.9	6.9	-4.5
5/2/93		-61.0	84.9	-172.5	158.1	214.5		-18.1	9.4	-4.0	1.4	3.8	-10.5
9/2/93		-49.3	100.8	-122.2	126.0	577.5		-15.8	12.4	-3.2	1.2	12.5	4.1
13/2/93		-66.0	119.8	-217.8	1520.6	-11.5		-19.8	13.9	-5.3	15.5	-0.2	-0.2
17/2/93		-96.9	201.3	47.2	233.5	-25.9		-26.2	22.8	1.2	2.2	-0.5	-7.3
21/2/93		-75.7	233.5	34.5	0.0	0.0		-21.6	29.4	1.0	0.0	0.0	2.4
25/2/93		-75.5	232.8	16.9	23.3	58.2		-22.0	30.1	0.5	0.2	1.2	3.3
1/3/93		-87.4	151.7	98.5	-22.6	45.2		-28.6	20.4	3.0	-0.2	1.0	-12.1
5/3/93		-87.9	190.8	-32.1	88.3	22.1		-28.7	27.0	-1.0	1.0	0.5	-9.9
9/3/93		-65.8	112.0	-23.5	43.1	10.8		-24.3	16.4	-0.8	0.5	0.3	-13.5
13/3/93		-60.4	151.1	29.5	20.3	60.9		-21.5	22.5	1.0	0.2	1.5	-1.3
17/3/93		-87.2	254.2	28.2	0.0	87.2		-28.9	40.4	1.0	0.0	2.3	3.0
21/3/93		-103.6	212.0	75.9	-166.9	52.2		-33.8	30.8	2.5	-2.0	1.2	-13.4
25/3/93		-107.7	212.7	35.6	-97.8	9.8		-37.5	33.8	1.3	-1.2	0.2	-19.3
29/3/93		-102.9	177.1	153.3	0.0	137.0		-33.9	24.8	5.0	0.0	3.3	-11.7
2/4/93		-87.0	180.9	-49.3	180.9	113.1		-27.4	24.1	-1.4	1.9	2.5	-7.8
6/4/93		-98.5	82.0	381.4	209.8	128.2		-30.3	9.4	10.6	2.2	2.7	-12.9
10/4/93		-83.8	40.4	263.4	614.6	0.0		-28.3	4.9	7.5	6.8	0.0	-15.8
14/4/93		-94.7	235.7	0.0	147.3	-73.6		-33.1	39.0	0.0	2.0	-1.9	-7.5
18/4/93		-93.3	98.2	661.8	285.1	-109.6		-32.0	13.0	23.8	3.9	-2.9	-4.2
22/4/93		-77.9	193.4	-24.0	-263.7	33.0		-26.4	28.7	-0.8	-3.0	0.8	-8.8
26/4/93		-55.3	144.4	-56.7	-66.8	55.7		-18.8	20.1	-1.7	-0.7	1.2	-3.8
Average		-86.3	169.2	64.0	159.3	69.8		-27.8	23.5	2.2	1.7	1.5	-7.5

APPENDIX 1 (cont.)

Date	NH3-N	Raw sewage	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5	NH3-N loading	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5
24/1/93	(mg/L)	12.9	1.8	0.8	0.5	0.2	0.2	(kg/d)	95.6	13.3	5.9	3.7	1.5
28/1/93		15.1	2	1.4	0.8	0.5	0.2		118.2	15.7	11.0	6.3	3.9
1/2/93		15.9	2.3	0.6	0.6	0.5	0.3		133.1	19.3	5.0	5.0	4.2
5/2/93		13.5	2.5	0.4	0.5	0.2	0.19		109.8	20.3	3.3	4.1	1.6
9/2/93		14.1	2.1	0.2	0.3	0.3	0.2		106.6	15.9	1.5	2.3	2.3
13/2/93		16.2	2	0.1	0.4	0.1	0.1		134.4	16.6	0.8	3.3	0.8
17/2/93		15.7	1.8	0.4	0.3	0.2	0.2		146.6	16.8	3.7	2.8	1.9
21/2/93		14.6	0	0.1	0.2	0.1	1		124.7	0.0	0.9	1.7	0.9
25/2/93		15	0.6	0	0.4	0.8	1.4		125.7	5.0	0.0	3.4	6.7
1/3/93		17.8	2.6	0.4	0.9	0.5	0.4		144.7	21.1	3.3	7.3	4.1
5/3/93		28.2	2.4	0.3	2.5	2.1	3.1		224.2	19.1	2.4	19.9	16.7
9/3/93		15.5	3.7	0.1	0.9	0.9	0.7		120.1	28.7	0.8	7.0	7.0
13/3/93		11.9	2.2	0	0	0.4	0.3		87.0	16.1	0.0	0.0	2.9
17/3/93		17.8	1.8	1.6	0.7	0.6	0.4		124.1	12.6	11.2	4.9	4.2
21/3/93		20	2.8	2.4	1.8	0.8	2		150.2	21.0	18.0	13.5	6.0
25/3/93		18.6	1.2	1.5	1	0.2	0.8		130.9	8.4	10.6	7.0	1.4
29/3/93		24.7	1.3	2	0.5	0	0		187.4	9.9	15.2	3.8	0.0
2/4/93		21.8	1.5	0.9	0.7	0.2	0.4		177.5	12.2	7.3	5.7	1.6
6/4/93		15.3	1.8	0.6	0	0	0		128.4	15.1	5.0	0.0	0.0
10/4/93		14.7	2.1	0.3	0.1	0	0.1		116.2	16.6	2.4	0.8	0.0
14/4/93		17	2.5	0.4	0.5	0.3	0.2		128.8	18.9	3.0	3.8	2.3
18/4/93		13.1	2.1	1.8	0.9	0.4	0.3		103.4	16.6	14.2	7.1	3.2
22/4/93		17.2	2	2.1	0.8	0.5	0.5		136.1	15.8	16.6	6.3	4.0
26/4/93		22.9	2	2.2	1	0.4	0		183.7	16.0	17.6	8.0	3.2
Average		17.1	2.0	0.9	0.7	0.4	0.5		134.9	15.5	6.7	5.3	3.3

APPENDIX 1 (cont.)

Date	NH3-N loading	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5	NH3-N removal	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5
24/1/93	(kg/ ha d)	7.2	3.0	6.0	10.3	2.1	(kg/ha d)	6.2	1.6	2.2	6.2	0.0
28/1/93		9.0	3.5	11.1	17.4	5.4		7.8	1.0	4.7	6.5	3.3
1/2/93		10.1	4.3	5.1	14.0	5.8		8.6	3.2	0.0	2.3	2.3
5/2/93		8.3	4.5	3.3	11.3	2.3		6.8	3.8	-0.8	6.8	0.1
9/2/93		8.1	3.5	1.5	6.3	3.2		6.9	3.2	-0.8	0.0	1.1
13/2/93		10.2	3.7	0.8	9.2	1.2		8.9	3.5	-2.5	6.9	0.0
17/2/93		11.1	3.7	3.8	7.8	2.6		9.8	2.9	0.9	2.6	0.0
21/2/93		9.4	0.0	0.9	4.7	1.2		9.4	-0.2	-0.9	2.4	-10.7
25/2/93		9.5	1.1	0.0	9.3	9.3		9.1	1.1	-3.4	-9.3	-7.0
1/3/93		11.0	4.7	3.3	20.3	5.6		9.4	4.0	-4.1	9.0	1.1
5/3/93		17.0	4.2	2.4	55.2	23.2		15.5	3.7	-17.7	8.8	-11.0
9/3/93		9.1	6.4	0.8	19.4	9.7		6.9	6.2	-6.3	0.0	2.2
13/3/93		6.6	3.6	0.0	0.0	4.1		5.4	3.6	0.0	-8.1	1.0
17/3/93		9.4	2.8	11.3	13.6	5.8		8.5	0.3	6.3	1.9	1.9
21/3/93		11.4	4.7	18.2	37.6	8.3		9.8	0.7	4.6	20.9	-12.5
25/3/93		9.9	1.9	10.7	19.6	2.0		9.3	-0.5	3.6	15.6	-5.9
29/3/93		14.2	2.2	15.3	10.5	0.0		13.5	-1.2	11.5	10.5	0.0
2/4/93		13.4	2.7	7.4	15.8	2.3		12.5	1.1	1.6	11.3	-2.3
6/4/93		9.7	3.4	5.1	0.0	0.0		8.6	2.2	5.1	0.0	0.0
10/4/93		8.8	3.7	2.4	2.2	0.0		7.5	3.2	1.6	2.2	-1.1
14/4/93		9.8	4.2	3.1	10.5	3.2		8.3	3.5	-0.8	4.2	1.1
18/4/93		7.8	3.7	14.4	19.7	4.4		6.6	0.5	7.2	11.0	1.1
22/4/93		10.3	3.5	16.8	17.6	5.5		9.1	-0.2	10.4	6.6	0.0
26/4/93		13.9	3.6	17.8	22.3	4.5		12.7	-0.4	9.7	13.4	4.5
Average		10.2	3.4	6.7	14.8	4.6		9.0	2.0	1.3	5.5	-1.3

APPENDIX 1 (cont.)

Date	NH3-N removal	Pond 1	Pond 2	Pond	Pond 4	Pond 5	Total	NO3-N	Raw sewage	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5
24/1/93	(%)	86.0	55.6	37.5	60.0	0.0	98.4	(mg/L)	0.5300	0.4000	0.2000	0.3700	0.3000	0.5000
28/1/93		86.8	30.0	42.9	37.5	60.0	98.7		0.3500	0.3200	0.1000	0.2000	0.2000	0.3800
1/2/93		85.5	73.9	0.0	16.7	40.0	98.1		0.4100	0.2900	0.3000	0.3200	0.2000	0.2000
5/2/93		81.5	84.0	-25.0	60.0	5.0	98.6		0.4100	0.2000	0.2000	0.4000	0.3800	0.6000
9/2/93		85.1	90.5	-50.0	0.0	33.3	98.6		0.5500	0.3000	0.5000	0.6000	0.2000	0.4000
13/2/93		87.7	95.0	-300.0	75.0	0.0	99.4		0.4500	0.3000	0.6000	0.4000	0.5000	0.6000
17/2/93		88.5	77.8	25.0	33.3	0.0	98.7		0.6100	0.3000	0.2000	0.5000	0.4000	0.5000
21/2/93		100.0	0.0	-100.0	50.0	-900.0	93.2		0.4500	0.7000	0.4000	0.6000	0.4200	0.1000
25/2/93		96.0	100.0	0.0	-100.0	-75.0	90.7		0.1300	0.6000	0.5000	0.3000	0.1000	0.0000
1/3/93		85.4	84.6	-125.0	44.4	20.0	97.8		0.1300	0.1400	0.2000	0.1000	0.1100	0.2000
5/3/93		91.5	87.5	-733.3	16.0	-47.6	89.0		0.0800	0.0600	0.3000	0.0600	0.0000	0.0000
9/3/93		76.1	97.3	-800.0	0.0	22.2	95.5		0.1000	0.0000	0.3000	0.1000	0.0800	0.0000
13/3/93		81.5	100.0	0.0	0.0	25.0	97.5		0.1300	0.1400	0.4000	0.4000	0.0800	0.1100
17/3/93		89.9	11.1	56.3	14.3	33.3	97.8		0.0800	0.2000	0.0800	0.1000	0.0900	0.0700
21/3/93		86.0	14.3	25.0	55.6	-150.0	90.0		0.0900	0.0900	0.0000	0.0800	0.0600	0.0000
25/3/93		93.5	-25.0	33.3	80.0	-300.0	95.7		0.0300	0.5000	0.1000	0.0100	0.3000	0.0200
29/3/93		94.7	-53.8	75.0	100.0	0.0	100.0		0.0300	0.4000	0.1200	0.2000	0.4000	0.4000
2/4/93		93.1	40.0	22.2	71.4	-100.0	98.2		0.0600	0.3000	0.0700	0.0800	0.2000	0.2000
6/4/93		88.2	66.7	100.0	0.0	0.0	100.0		0.0900	0.3000	0.0900	0.4000	0.3000	0.4000
10/4/93		85.7	85.7	66.7	100.0	0.0	99.3		0.0800	0.0900	0.3000	0.4000	0.3000	0.3000
14/4/93		85.3	84.0	-25.0	40.0	33.3	98.8		0.0900	0.0000	0.2000	0.1000	0.1000	0.2000
18/4/93		84.0	14.3	50.0	55.6	25.0	97.7		0.0400	0.0600	0.1000	0.0900	0.0900	0.2000
22/4/93		88.4	-5.0	61.9	37.5	0.0	97.1		0.0700	0.1000	0.0000	0.1000	0.1100	0.0000
26/4/93		91.3	-10.0	54.5	60.0	100.0	100.0		0.0900	0.2000	0.0700	0.1000	0.0900	0.1000
Average		88.0	49.9	-62.8	37.8	-49.0	97.0		0.2117	0.2496	0.2221	0.2504	0.2088	0.2283

APPENDIX 1 (cont.)

Date	NO3-N loading	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5	NO3-N loading	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5
24/1/93	(kg/d)	3.9	3.0	1.5	2.7	2.2	(kg/ha d)	0.3	0.7	1.5	7.6	3.1
28/1/93		2.7	2.5	0.8	1.6	1.6		0.2	0.6	0.8	4.3	2.2
1/2/93		3.4	2.4	2.5	2.7	1.7		0.3	0.5	2.5	7.4	2.3
5/2/93		3.3	1.6	1.6	3.3	3.1		0.3	0.4	1.6	9.0	4.3
9/2/93		4.2	2.3	3.8	4.5	1.5		0.3	0.5	3.8	12.6	2.1
13/2/93		3.7	2.5	5.0	3.3	4.1		0.3	0.6	5.0	9.2	5.8
17/2/93		5.7	2.8	1.9	4.7	3.7		0.4	0.6	1.9	13.0	5.2
21/2/93		3.8	6.0	3.4	5.1	3.6		0.3	1.3	3.5	14.2	5.0
25/2/93		1.1	5.0	4.2	2.5	0.8		0.1	1.1	4.2	7.0	1.2
1/3/93		1.1	1.1	1.6	0.8	0.9		0.1	0.3	1.6	2.3	1.2
5/3/93		0.6	0.5	2.4	0.5	0.0		0.0	0.1	2.4	1.3	0.0
9/3/93		0.8	0.0	2.3	0.8	0.6		0.1	0.0	2.3	2.2	0.9
13/3/93		1.0	1.0	2.9	2.9	0.6		0.1	0.2	3.0	8.1	0.8
17/3/93		0.6	1.4	0.6	0.7	0.6		0.0	0.3	0.6	1.9	0.9
21/3/93		0.7	0.7	0.0	0.6	0.5		0.1	0.2	0.0	1.7	0.6
25/3/93		0.2	3.5	0.7	0.1	2.1		0.0	0.8	0.7	0.2	2.9
29/3/93		0.2	3.0	0.9	1.5	3.0		0.0	0.7	0.9	4.2	4.2
2/4/93		0.5	2.4	0.6	0.7	1.6		0.0	0.5	0.6	1.8	2.3
6/4/93		0.8	2.5	0.8	3.4	2.5		0.1	0.6	0.8	9.3	3.5
10/4/93		0.6	0.7	2.4	3.2	2.4		0.0	0.2	2.4	8.8	3.3
14/4/93		0.7	0.0	1.5	0.8	0.8		0.1	0.0	1.5	2.1	1.1
18/4/93		0.3	0.5	0.8	0.7	0.7		0.0	0.1	0.8	2.0	1.0
22/4/93		0.6	0.8	0.0	0.8	0.9		0.0	0.2	0.0	2.2	1.2
26/4/93		0.7	1.6	0.6	0.8	0.7		0.1	0.4	0.6	2.2	1.0
Average		1.7	2.0	1.8	2.0	1.7		0.1	0.4	1.8	5.6	2.3

APPENDIX 1 (cont.)

Date	NO3-N removal	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5	NO3-N removal	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5	Total
24/1/93	(kg/ha d)	0.1	0.3	-1.3	1.4	-2.1	(%)	24.5	50.0	-85.0	18.9	-66.7	5.66
28/1/93		0.0	0.4	-0.8	0.0	-2.0		8.6	68.8	-100.0	0.0	-90.0	-8.57
1/2/93		0.1	0.0	-0.2	2.8	0.0		29.3	-3.4	-6.7	37.5	0.0	51.22
5/2/93		0.1	0.0	-1.6	0.5	-2.5		51.2	0.0	-100.0	5.0	-57.9	-46.34
9/2/93		0.1	-0.3	-0.8	8.4	-2.1		45.5	-66.7	-20.0	66.7	-100.0	27.27
13/2/93		0.1	-0.6	1.7	-2.3	-1.2		33.3	-100.0	33.3	-25.0	-20.0	-33.33
17/2/93		0.2	0.2	-2.8	2.6	-1.3		50.8	33.3	-150.0	20.0	-25.0	18.03
21/2/93		-0.2	0.6	-1.7	4.3	3.8		-55.6	42.9	-50.0	30.0	76.2	77.78
25/2/93		-0.3	0.2	1.7	4.7	1.2		-361.5	16.7	40.0	66.7	100.0	100.00
1/3/93		0.0	-0.1	0.8	-0.2	-1.0		-7.7	-42.9	50.0	-10.0	-81.8	-53.85
5/3/93		0.0	-0.4	1.9	1.3	0.0		25.0	-400.0	80.0	100.0	0.0	100.00
9/3/93		0.1	-0.5	1.6	0.4	0.9		100.0	0.0	66.7	20.0	100.0	100.00
13/3/93		0.0	-0.4	0.0	6.5	-0.3		-7.7	-185.7	0.0	80.0	-37.5	15.38
17/3/93		-0.1	0.2	-0.1	0.2	0.2		-150.0	60.0	-25.0	10.0	22.2	12.50
21/3/93		0.0	0.2	-0.6	0.4	0.6		0.0	100.0	0.0	25.0	100.0	100.00
25/3/93		-0.3	0.6	0.6	-5.7	2.7		-1566.7	80.0	90.0	-2900.0	93.3	33.33
29/3/93		-0.2	0.5	-0.6	-4.2	0.0		-1233.3	70.0	-66.7	-100.0	0.0	-1233.33
2/4/93		-0.1	0.4	-0.1	-2.7	0.0		-400.0	76.7	-14.3	-150.0	0.0	-233.33
6/4/93		-0.1	0.4	-2.6	2.3	-1.2		-233.3	70.0	-344.4	25.0	-33.3	-344.44
10/4/93		0.0	-0.4	-0.8	2.2	0.0		-12.5	-233.3	-33.3	25.0	0.0	-275.00
14/4/93		0.1	-0.3	0.8	0.0	-1.1		100.0	0.0	50.0	0.0	-100.0	-122.22
18/4/93		0.0	-0.1	0.1	0.0	-1.2		-50.0	-66.7	10.0	0.0	-122.2	-400.00
22/4/93		0.0	0.2	-0.8	-0.2	1.2		-42.9	100.0	0.0	-10.0	100.0	100.00
26/4/93		-0.1	0.2	-0.2	0.2	-0.1		-122.2	65.0	-42.9	10.0	-11.1	-11.11
Average		0.0	0.0	-0.2	1.0	-0.2		-157.3	-11.1	-25.8	-110.6	-6.4	-84.2

APPENDIX 1 (cont.)

Date	NO2-N	Raw sewage	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5	NO2-N loading	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5
24/1/93	(mg/L)	0.02	0.20	0.80	0.20	0.20	0.20	(kg/d)	0.1	1.5	5.9	1.5	1.5
28/1/93		0.02	0.50	1,20	0.70	0.60	0.20		0.2	3.9	9.4	5.5	4.7
1/2/93		0.03	0.70	0.40	0.54	0.36	0.25		0.3	5.9	3.3	4.5	3.0
5/2/93		0.03	0.80	0.20	0.73	0.30	0.20		0.2	6.5	1.6	5.9	2.4
9/2/93		0.03	0.40	0.10	0.50	0.41	0.27		0.2	3.0	0.8	3.8	3.1
13/2/93		0.02	0.40	0.10	0.60	0.20	0.21		0.2	3.3	0.8	5.0	1.7
17/2/93		0.02	0.30	0.20	0.30	0.20	0.19		0.2	2.8	1.9	2.8	1.9
21/2/93		0.02	0.10	0.10	0.20	0.20	0.30		0.2	0.9	0.9	1.7	1.7
25/2/93		0.02	0.20	0.10	0.30	0.60	0.70		0.2	1.7	0.8	2.5	5.0
1/3/93		0.30	0.60	0.30	0.80	0.57	0.35		2.4	4.9	2.4	6.5	4.6
5/3/93		0.02	0.40	0.20	0.90	0.70	0.80		0.2	3.2	1.6	7.2	5.6
9/3/93		0.02	0.90	0.20	0.42	0.60	0.50		0.2	7.0	1.6	3.3	4.7
13/3/93		0.03	0.30	0.20	0.20	0.30	0.20		0.2	2.2	1.5	1.5	2.2
17/3/93		0.04	0.22	1.20	0.62	0.60	0.20		0.3	1.5	8.4	4.3	4.2
21/3/93		0.02	0.70	1.70	0.90	0.72	0.75		0.2	5.3	12.8	6.8	5.4
25/3/93		0.03	0.10	1.20	0.80	0.30	0.54		0.2	0.7	8.4	5.6	2.1
29/3/93		0.04	0.20	1.40	0.27	0.20	0.10		0.3	1.5	10.6	2.0	1.5
2/4/93		0.05	0.21	0.60	0.70	0.27	0.25		0.4	1.7	4.9	5.7	2.2
6/4/93		0.04	0.20	0.30	0.20	0.20	0.10		0.3	1.7	2.5	1.7	1.7
10/4/93		0.02	0.30	0.20	0.20	0.20	0.10		0.2	2.4	1.6	1.6	1.6
14/4/93		0.02	0.40	0.30	0.50	0.20	0.20		0.2	3.0	2.3	3.8	1.5
18/4/93		0.02	0.22	1.10	0.80	0.41	0.20		0.2	1.7	8.7	6.3	3.2
22/4/93		0.03	0.20	1.40	0.60	0.45	0.30		0.2	1.6	11.1	4.7	3.6
26/4/93		0.01	0.20	1.00	0.90	0.30	0.10		0.1	1.6	8.0	7.2	2.4
Average		0.04	0.36	0.60	0.54	0.38	0.30		0.3	2.9	4.7	4.2	3.0

APPENDIX 1 (cont.)

Date	NO2-N loading	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5	NO2-N removal	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5
24/1/93	(kg/ha d)	0.0	0.3	6.0	4.1	2.1	(kg/ha d)	-0.1	-1.0	4.5	0.0	0.0
28/1/93		0.0	0.9	9.5	15.2	6.5		-0.3	-1.2	4.0	2.2	4.3
1/2/93		0.0	1.3	3.4	12.6	4.2		-0.4	0.6	-1.2	4.2	1.3
5/2/93		0.0	1.4	1.6	16.5	3.4		-0.5	1.1	-4.4	9.7	1.1
9/2/93		0.0	0.7	0.8	10.5	4.3		-0.2	0.5	-3.1	1.9	1.5
13/2/93		0.0	0.7	0.8	13.8	2.3		-0.2	0.6	-4.2	9.2	-0.1
17/2/93		0.0	0.6	1.9	7.8	2.6		-0.2	0.2	-0.9	2.6	0.1
21/2/93		0.0	0.2	0.9	4.7	2.4		-0.1	0.0	-0.9	0.0	-1.2
25/2/93		0.0	0.4	0.8	7.0	7.0		-0.1	0.2	-1.7	-7.0	-1.2
1/3/93		0.2	1.1	2.5	18.1	6.4		-0.2	0.5	-4.1	5.2	2.5
5/3/93		0.0	0.7	1.6	19.9	7.7		-0.2	0.4	-5.6	4.4	-1.1
9/3/93		0.0	1.6	1.6	9.0	6.5		-0.5	1.2	-1.7	-3.9	1.1
13/3/93		0.0	0.5	1.5	4.1	3.0		-0.1	0.2	0.0	-2.0	1.0
17/3/93		0.0	0.3	8.5	12.0	5.8		-0.1	-1.5	4.1	0.4	3.9
21/3/93		0.0	1.2	12.9	18.8	7.5		-0.4	-1.7	6.1	3.8	-0.3
25/3/93		0.0	0.2	8.5	15.6	2.9		0.0	-1.7	2.8	9.8	-2.3
29/3/93		0.0	0.3	10.7	5.7	2.1		-0.1	-2.0	8.7	1.5	1.1
2/4/93		0.0	0.4	4.9	15.8	3.1		-0.1	-0.7	-0.8	9.7	0.2
6/4/93		0.0	0.4	2.5	4.7	2.3		-0.1	-0.2	0.8	0.0	1.2
10/4/93		0.0	0.5	1.6	4.4	2.2		-0.2	0.2	0.0	0.0	1.1
14/4/93		0.0	0.7	2.3	10.5	2.1		-0.2	0.2	-1.5	6.3	0.0
18/4/93		0.0	0.4	8.8	17.5	4.5		-0.1	-1.5	2.4	8.6	2.3
22/4/93		0.0	0.4	11.2	13.2	4.9		-0.1	-2.1	6.4	3.3	1.6
26/4/93		0.0	0.4	8.1	20.1	3.3		-0.1	-1.4	0.8	13.4	2.2
Average		0.0	0.6	4.7	11.7	4.1		-0.2	-0.4	0.4	3.5	0.8

APPENDIX 1 (cont.)

Date	NO2-N removal	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5	PO4-P	Raw sewage	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5
24/1/93	(%)	-900.0	-300.0	75.0	0.0	0.0	(mg/L)	15.3	2.4	1.2	1.1	0.0	0.0
28/1/93		-2400.0	-140.0	41.7	14.3	66.7		13.2	2.8	1.5	0.5	0.1	0.0
1/2/93		-2233.3	42.9	-35.0	33.3	30.6		14.5	2.7	1.2	0.8	0.0	0.0
5/2/93		-2566.7	75.0	-265.0	58.9	33.3		14.9	2.3	0.3	0.0	0.2	0.3
9/2/93		-1233.3	75.0	-400.0	18.0	34.1		16.7	2.4	0.4	0.0	0.1	0.0
13/2/93		-1900.0	75.0	-500.0	66.7	-5.0		15.4	2.7	0.7	0.0	0.0	0.0
17/2/93		-1400.0	33.3	-50.0	33.3	5.0		13.6	2.2	0.7	0.0	0.0	0.0
21/2/93		-400.0	0.0	-100.0	0.0	-50.0		14.0	2.1	0.8	0.2	0.0	0.0
25/2/93		-900.0	50.0	-200.0	-100.0	-16.7		14.4	2.3	1.0	0.2	0.0	0.0
1/3/93		-100.0	50.0	-166.7	28.8	38.6		15.6	2.8	1.1	0.5	0.0	0.0
5/3/93		-1900.0	50.0	-350.0	22.2	-14.3		16.7	2.7	1.3	0.3	0.0	0.0
9/3/93		-4400.0	77.8	-110.0	-42.9	16.7		15.5	2.7	1.4	0.9	0.0	0.0
13/3/93		-900.0	33.3	0.0	-50.0	33.3		19.2	2.5	0.9	0.3	0.0	0.0
17/3/93		-450.0	-445.5	48.3	3.2	66.7		15.6	2.6	0.8	1.0	0.4	0.0
21/3/93		-3400.0	-142.9	47.1	20.0	-4.2		16.3	2.6	0.9	0.2	0.0	0.0
25/3/93		-233.3	-1100.0	33.3	62.5	-80.0		17.8	2.5	1.0	0.9	0.0	0.0
29/3/93		-400.0	-600.0	80.7	25.9	50.0		21.1	2.6	0.9	0.7	0.1	0.0
2/4/93		-320.0	-185.7	-16.7	61.4	7.4		17.8	2.5	0.7	0.3	0.2	0.0
6/4/93		-400.0	-50.0	33.3	0.0	50.0		16.0	2.7	0.2	0.5	0.0	0.0
10/4/93		-1400.0	33.3	0.0	0.0	50.0		19.6	2.9	0.7	0.1	0.7	0.1
14/4/93		-1900.0	25.0	-66.7	60.0	0.0		17.8	2.8	1.7	0.8	0.0	0.0
18/4/93		-1000.0	-400.0	27.3	48.8	51.2		21.4	2.3	0.5	0.5	0.0	0.0
22/4/93		-566.7	-600.0	57.1	25.0	33.3		16.7	2.4	1.2	0.4	0.0	0.0
26/4/93		-1900.0	-400.0	10.0	66.7	66.7		17.0	2.2	0.8	0.3	0.1	0.0
Average		-1383.5	-156.0	-75.3	19.0	19.3		16.5	2.5	0.9	0.4	0.1	0.0

APPENDIX 1 (cont.)

Date	PO4-P loading	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5	PO4-P loading	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5
24/1/93	(kg/d)	113.4	17.8	8.9	8.2	0.0	(kg/ha d)	8.6	4.0	9.0	22.6	0.0
28/1/93		103.3	21.9	11.7	3.9	0.8		7.8	4.9	11.9	10.9	1.1
1/2/93		121.4	22.6	10.0	6.7	0.0		9.2	5.0	10.2	18.6	0.0
5/2/93		121.1	18.7	2.4	0.0	1.6		9.2	4.2	2.5	0.0	2.3
9/2/93		126.3	18.1	3.0	0.0	0.8		9.6	4.0	3.1	0.0	1.1
13/2/93		127.7	22.4	5.8	0.0	0.0		9.7	5.0	5.9	0.0	0.0
17/2/93		127.0	20.5	6.5	0.0	0.0		9.6	4.6	6.6	0.0	0.0
21/2/93		119.6	17.9	6.8	1.7	0.0		9.1	4.0	6.9	4.7	0.0
25/2/93		120.7	19.3	8.4	1.7	0.0		9.1	4.3	8.5	4.7	0.0
1/3/93		126.8	22.8	8.7	4.1	0.0		9.6	5.1	8.8	11.3	0.0
5/3/93		132.7	21.5	10.3	2.4	0.0		10.1	4.8	10.4	6.6	0.0
9/3/93		120.1	20.9	10.9	7.0	0.0		9.1	4.7	11.0	19.4	0.0
13/3/93		140.3	18.3	6.6	2.2	0.0		10.6	4.1	6.6	6.1	0.0
17/3/93		108.8	18.1	5.6	7.0	2.8		8.2	4.0	5.6	19.4	3.9
21/3/93		122.4	19.5	6.8	1.5	0.0		9.3	4.3	6.8	4.2	0.0
25/3/93		125.3	17.6	7.0	6.3	0.0		9.5	3.9	7.1	17.6	0.0
29/3/93		160.1	19.7	6.8	5.3	0.8		12.1	4.4	6.9	14.8	1.1
2/4/93		144.9	20.4	5.7	2.4	1.6		11.0	4.5	5.8	6.8	2.3
6/4/93		134.3	22.7	1.7	4.2	0.0		10.2	5.0	1.7	11.7	0.0
10/4/93		154.9	22.9	5.5	0.8	5.5		11.7	5.1	5.6	2.2	7.7
14/4/93		134.8	21.2	12.9	6.1	0.0		10.2	4.7	13.0	16.8	0.0
18/4/93		168.9	18.2	3.9	3.9	0.0		12.8	4.0	4.0	11.0	0.0
22/4/93		132.1	19.0	9.5	3.2	0.0		10.0	4.2	9.6	8.8	0.0
26/4/93		136.4	17.6	6.4	2.4	0.8		10.3	3.9	6.5	6.7	1.1
Average		130.1	20.0	7.2	3.4	0.6		9.9	4.4	7.2	9.4	0.8

APPENDIX 1 (cont.)

Date	PO4-P removal	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5	PO4-P removal	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5
24/1/93	(kg/ha d)	7.2	2.0	0.7	22.6	0.0	(%)	84.3	50.0	8.3	100.0	0.0
28/1/93		6.2	2.3	7.9	8.7	1.1		78.8	46.4	66.7	80.0	100.0
1/2/93		7.5	2.8	3.4	18.6	0.0		81.4	55.6	33.3	100.0	0.0
5/2/93		7.8	3.6	2.5	-4.5	-1.5		84.6	87.0	100.0	0.0	-65.0
9/2/93		8.2	3.4	3.1	-2.1	1.1		85.6	83.3	100.0	0.0	100.0
13/2/93		8.0	3.7	5.9	0.0	0.0		82.5	74.1	100.0	0.0	0.0
17/2/93		8.1	3.1	6.6	0.0	0.0		83.8	68.2	100.0	0.0	0.0
21/2/93		7.7	2.5	5.2	4.7	0.0		85.0	61.9	75.0	100.0	0.0
25/2/93		7.7	2.4	6.8	4.7	0.0		84.0	56.5	80.0	100.0	0.0
1/3/93		7.9	3.1	4.7	11.3	0.0		82.1	61.8	53.3	100.0	0.0
5/3/93		8.4	2.5	8.0	6.6	0.0		83.8	51.9	76.9	100.0	0.0
9/3/93		7.5	2.2	3.9	19.4	0.0		82.6	48.1	35.7	100.0	0.0
13/3/93		9.2	2.6	4.4	6.1	0.0		87.0	64.0	66.7	100.0	0.0
17/3/93		6.9	2.8	-1.4	11.6	3.9		83.3	69.2	-25.0	60.0	100.0
21/3/93		7.8	2.8	5.3	4.2	0.0		84.0	65.4	77.8	100.0	0.0
25/3/93		8.2	2.3	0.7	17.6	0.0		86.0	60.0	10.0	100.0	0.0
29/3/93		10.6	2.9	1.5	12.6	1.1		87.7	65.4	22.2	85.7	100.0
2/4/93		9.4	3.3	3.3	2.3	2.3		86.0	72.0	57.1	33.3	100.0
6/4/93		8.5	4.7	-2.5	11.7	0.0		83.1	92.6	-150.0	100.0	0.0
10/4/93		10.0	3.9	4.8	-13.2	6.3		85.2	75.9	85.7	-600.0	81.4
14/4/93		8.6	1.9	6.9	16.8	0.0		84.3	39.3	52.9	100.0	0.0
18/4/93		11.4	3.2	0.0	11.0	0.0		89.3	78.3	0.0	100.0	0.0
22/4/93		8.6	2.1	6.4	8.8	0.0		85.6	50.0	66.7	100.0	0.0
26/4/93		9.0	2.5	4.1	4.5	1.1		87.1	63.6	62.5	66.7	100.0
Average		8.3	2.8	3.8	7.7	0.6		84.5	64.2	48.2	46.9	25.7

APPENDIX 1 (cont.)

Date	DO	Raw sewage	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5	pH	Raw sewage	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5
24/1/93	(mg/L)	0.0	21.0	17.0	15.0	12.0	8.0		7.4	9.2	8.7	8.8	8.5	8.4
28/1/93		0.0	16.0	15.0	12.0	9.0	6.0		7.3	9	8.1	8.6	8.4	7.6
1/2/93		0.0	15.0	14.0	11.0	8.0	5.0		7.7	9	8.5	8.6	8.2	7.4
5/2/93		0.0	21.0	20.0	15.0	14.0	10.0		7.6	9.3	9.4	9.1	8.9	8.8
9/2/93		0.0	21.0	19.0	16.0	13.0	9.0		7.3	9.3	9.2	9	8.7	8.6
13/2/93		0.0	20.0	19.0	16.0	13.0	8.0		7.5	9.1	9	8.9	8.7	8.3
17/2/93		0.0	21.0	17.0	15.0	12.0	7.0		7.6	9.4	8.8	8.8	8.5	8.2
21/2/93		0.0	23.0	17.0	13.0	10.0	7.0		7.4	9.6	8.7	8.5	8.5	8.5
25/2/93		0.0	21.0	17.0	15.0	9.0	7.0		7.4	9.4	8.6	8.5	8.4	8.2
1/3/93		0.0	16.0	16.0	13.0	8.0	7.0		7.4	9	8.6	8.3	8.4	8.2
5/3/93		0.0	17.0	16.0	14.0	8.0	6.0		7.3	9.1	8.5	8.4	8.2	7.9
9/3/93		0.0	16.0	15.0	12.0	6.0	5.0		7.4	9	8.4	8.2	8.2	7.8
13/3/93		0.0	17.0	17.0	14.0	9.0	6.0		7.3	9.2	8.6	8.5	8.3	8
17/3/93		0.0	19.0	16.0	12.0	7.0	6.0		7.3	9.3	8.5	8.2	8.2	7.8
21/3/93		0.0	18.0	17.0	16.0	9.0	7.0		7.4	9.2	8.6	8.8	8.4	8.2
25/3/93		0.0	21.0	15.0	11.0	7.0	6.0		8.3	9.3	8.4	8.1	8.2	8.3
29/3/93		0.0	19.0	17.0	12.0	8.0	5.0		7.8	9.2	8.6	8.1	8.4	7.6
2/4/93		0.0	17.0	18.0	13.0	8.0	6.0		7.7	9.2	8.7	8.4	8.4	8.2
6/4/93		0.0	16.0	19.0	13.0	10.0	6.0		7.3	9.1	9.2	8.2	8.7	8.3
10/4/93		0.0	15.0	18.0	16.0	9.0	7.0		7.5	8.9	8.7	8.7	8.5	8.4
14/4/93		0.0	16.0	14.0	11.0	7.0	5.0		7.4	8.9	8.1	8.1	8.5	7.6
18/4/93		0.0	18.0	17.0	10.0	6.0	4.0		7.3	9.3	8.9	8.1	8.1	7.6
22/4/93		0.0	20.0	16.0	11.0	7.0	6.0		7.3	9.3	8.4	8.2	8.4	7.7
26/4/93		0.0	21.0	16.0	12.0	9.0	6.0		7.4	9.4	8.7	8.5	8.3	8.5
Average		0.0	18.5	16.8	13.3	9.1	6.5		7.5	9.2	8.7	8.5	8.4	8.1

APPENDIX 1 (cont.)

Date	Temperature	Ambient	Raw sewage	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5	TC	Raw sewage	FP effluent	MP effluent
24/1/93	(degree Celsius)	25.1	20.0	24.0	24.2	20.0	20.0	21.3	(MPN/100ml)	1590000	347	49
28/1/93		25.4	20.5	24.3	23.4	23.3	23.2	23.4		1410000	860	133
1/2/93		25.2	21.3	25.0	25.0	23.0	22.0	21.2		1370000	855	134
5/2/93		26.4	23.0	24.0	23.1	22.0	22.2	23.0		2050000	125	17
9/2/93		26.3	23.4	24.2	24.6	24.5	24.3	23.2		4500000	279	60
13/2/93		27.0	24.5	25.6	24.2	24.0	23.7	23.5		13900000	357	81
17/2/93		26.8	26.0	24.3	25.5	25.0	24.6	24.1		40000000	920	140
21/2/93		30.3	27.2	24.1	25.4	25.4	25.4	25.3		13500000	750	72
25/2/93		30.0	26.3	24.6	26.2	26.3	26.3	26.2		116000000	580	65
1/3/93		31.0	26.0	27.2	26.5	26.7	26.7	26.8		125000000	880	63
5/3/93		29.2	26.5	26.5	26.5	27.2	27.1	27.2		136000000	950	335
9/3/93		30.1	27.0	28.0	27.0	27.4	27.4	27.6		14900000	3700	155
13/3/93		30.5	27.2	27.8	27.8	27.8	27.8	28.0		15000000	2500	93
17/3/93		31.9	27.3	27.3	27.7	27.9	27.9	28.1		90000000	701	385
21/3/93		32.5	27.4	29.2	31.0	31.8	32.0	32.5		74000000	530	67
25/3/93		32.1	27.1	27.1	28.2	28.2	28.3	28.3		91000000	2350	55
29/3/93		30.2	27.2	27.2	28.0	28.5	28.5	28.1		1250000	700	152
2/4/93		30.4	27.4	28.4	28.2	28.3	28.3	28.4		9750000	437	93
6/4/93		29.8	27.3	27.4	27.4	27.8	28.0	27.7		7150000	290	91
10/4/93		30.2	27.2	28.2	27.8	27.9	28.3	28.2		13400000	530	70
14/4/93		30.1	27.2	28.3	28.1	28.3	28.3	28.4		15000000	4000	185
18/4/93		30.0	27.0	27.8	28.0	28.1	28.1	28.2		19000000	531	187
22/4/93		30.2	27.1	27.4	27.3	27.3	27.3	27.3		25000000	2350	112
26/4/93		30.4	27.0	27.2	27.2	27.2	27.3	27.2		14200000	811	103
Average		29.2	25.8	26.5	26.6	26.4	26.4	26.4		35207083.3	1097.2	120.7

APPENDIX 1 (cont.)

Date	TC removal (%)	FP effluent	MP effluent	Total	FC (MPN/ 100 ml)	Raw sewage	FP effluent	MP effluent	FC removal (%)	FP effluent	MP effluent	Total
24/1/93		99.98	85.88	100.00		410000	89	21		99.98	76.40	99.99
28/1/93		99.94	84.53	99.99		370000	100	54		99.97	46.00	99.99
1/2/93		99.94	84.33	99.99		550000	432	64		99.92	85.19	99.99
5/2/93		99.99	86.40	100.00		1510000	10	5		100.00	50.00	100.00
9/2/93		99.99	78.49	100.00		1250000	34	9		100.00	73.53	100.00
13/2/93		100.00	77.31	100.00		9500000	75	15		100.00	80.00	100.00
17/2/93		100.00	84.78	100.00		9150000	28	18		100.00	35.71	100.00
21/2/93		99.99	90.40	100.00		9750000	93	13		100.00	86.02	100.00
25/2/93		100.00	88.79	100.00		95000000	75	51		100.00	32.00	100.00
1/3/93		100.00	92.84	100.00		121750000	371	60		100.00	83.83	100.00
5/3/93		100.00	64.74	100.00		1310000	735	80		99.94	89.12	99.99
9/3/93		99.98	95.81	100.00		9000000	2400	94		99.97	96.08	100.00
13/3/93		99.98	96.28	100.00		1250000	1600	20		99.87	98.75	100.00
17/3/93		100.00	45.08	100.00		17900000	652	84		100.00	87.12	100.00
21/3/93		100.00	87.36	100.00		1250000	400	29		99.97	92.75	100.00
25/3/93		100.00	97.66	100.00		1130000	377	12		99.97	96.82	100.00
29/3/93		99.94	78.29	99.99		850000	500	111		99.94	77.80	99.99
2/4/93		100.00	78.72	100.00		1480000	432	49		99.97	88.66	100.00
6/4/93		100.00	68.62	100.00		1170000	275	35		99.98	87.27	100.00
10/4/93		100.00	86.79	100.00		9500000	605	25		99.99	95.87	100.00
14/4/93		99.97	95.38	100.00		12500000	912	107		99.99	88.27	100.00
18/4/93		100.00	64.78	100.00		1390000	400	89		99.97	77.75	99.99
22/4/93		99.99	95.23	100.00		1270000	651	75		99.95	88.48	99.99
26/4/93		99.99	87.30	100.00		1350000	348	38		99.97	89.08	100.00
Average		99.99	83.16	100.00		12941250.0	483.1	48.3		99.97	79.27	100.00

APPENDIX 2. Diurnal variation investigation data.

Time	Light intensity (K lux)	Temperature	Ambient	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5
6:00 AM	3.16	degrees Celsius	25.80	28.00	28.50	28.50	28.50	28.50
7:00 AM	11.93		27.50	28.50	28.50	28.50	28.50	28.50
8:00 AM	20.31		29.80	29.00	29.00	29.00	29.00	29.00
9:00 AM	32.82		32.50	30.00	29.50	29.50	29.50	29.50
10:00 AM	50.21		36.00	31.50	30.00	31.00	31.50	31.00
11:00 AM	53.20		38.30	32.50	32.50	32.50	32.50	32.00
12:00 AM	56.40		38.50	34.50	32.50	32.50	33.00	32.50
1:00 PM	50.90		38.60	36.50	35.00	34.00	35.00	34.00
2:00 PM	38.53		38.70	37.00	35.50	33.00	35.00	34.00
3:00 PM	29.17		38.50	37.00	36.00	33.00	33.50	33.50
4:00 PM	18.67		38.40	35.00	35.50	34.00	33.00	33.00
5:00 PM	12.54		37.20	33.50	35.00	32.50	33.00	32.00
6:00 PM	9.25		33.50	32.00	33.00	31.00	32.00	31.00

Time	PO4-P	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5	NO3-N	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5	NO2-N	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5
6:00 AM	(mg/L)	3.541	2.520	1.680	0.974	0.432	(mg/L)	0.127	0.102	0.100	0.099	0.108	(mg/L)	0.089	0.376	0.261	1.880	0.571
7:00 AM																		
8:00 AM																		
9:00 AM																		
10:00 AM																		
11:00 AM																		
12:00 AM		2.224	1.271	0.514	0.316	0.040		0.311	0.210	0.138	0.114	0.135		0.176	1.880	0.175	0.114	0.472
1:00 PM																		
2:00 PM																		
3:00 PM																		
4:00 PM																		
5:00 PM																		
6:00 PM		1.947	1.130	0.433	0.212	0.037		0.182	0.139	0.101	0.100	0.119		0.000	0.188	0.422	0.287	0.459

APPENDIX 2 (cont.)

Time	pH	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5	DO	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5
6:00 AM		7.70	7.50	7.40	7.30	7.10	(mg/L)	11.00	6.20	5.60	5.00	4.10
7:00 AM		7.80	7.60	7.60	7.50	7.40		12.50	6.30	5.70	5.50	5.30
8:00 AM		7.90	7.80	7.70	7.70	7.50		14.30	9.00	8.50	7.90	7.60
9:00 AM		8.40	8.20	8.10	7.90	7.70		16.20	10.70	9.60	9.30	9.10
10:00 AM		8.70	8.50	8.30	8.20	8.00		18.80	15.70	13.20	12.80	11.20
11:00 AM		9.20	8.80	8.60	8.40	8.20		19.00	15.90	14.80	13.20	12.80
12:00 AM		9.30	8.90	8.70	8.50	8.30		20.10	17.10	15.70	14.70	14.20
1:00 PM		9.00	8.80	8.60	8.40	8.20		19.20	16.80	15.40	14.50	13.90
2:00 PM		9.10	8.80	8.50	8.20	8.10		18.10	16.10	14.80	14.10	13.50
3:00 PM		9.00	8.70	8.40	8.20	8.00		17.70	15.70	14.40	13.80	12.70
4:00 PM		8.90	8.70	8.40	8.20	8.00		17.50	15.60	13.90	11.90	10.70
5:00 PM		8.70	8.40	8.30	8.10	7.90		16.40	14.80	13.50	10.20	9.90
6:00 PM		8.60	8.40	8.20	8.10	7.80		15.80	14.30	13.40	9.80	8.70

Time	NH3-N	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5	H2S	Pond 1	Pond 2	Pond 3	Pond 4	Pond 5
6:00 AM	(mg/L)	17.125	2.113	2.044	1.951	1.562	(mg/L)	0	0	0	0	0
7:00 AM												
8:00 AM												
9:00 AM												
10:00 AM												
11:00 AM												
12:00 AM		2.040	0.000	0.201	0.000	0.000		0	0	0	0	0
1:00 PM												
2:00 PM												
3:00 PM												
4:00 PM												
5:00 PM												
6:00 PM		0.000	0.000	0.000	0.000	0.000		0	0	0	0	0

REFERENCES

ABELIOVICH, A., 1982; Biological Equilibrium in a Wastewater Reservoir, *Water Research*, **16**, 1135-1138.

AHMED, S.M.R., 1981; Choice and Layout of Various Types of Stabilisation Ponds, in *Waste Stabilisation Ponds Design and Operation*, 133-154; A Report of a Seminar at Lahore, 29 December 1979-5 January 1980, World Health Organisation, Geneva.

ALLUM, M.O., 1955; Lagoon Purification in South Dakota, *The American City*, **70**, 128-129.

AMERICAN PUBLIC HEALTH ASSOCIATION, AMERICAN WATER WORKS ASSOCIATION AND WATER POLLUTION CONTROL FEDERATION, 1985; *Standard Methods for the Examination of Water and Wastewater*; Sixteenth Edition, American Public Health Association, Washington DC.

AMERICAN PUBLIC HEALTH ASSOCIATION, AMERICAN WATER WORKS ASSOCIATION AND WATER POLLUTION CONTROL FEDERATION, 1989; *Standard Methods for the Examination of Water and Wastewater*; Seventeenth Edition, American Public Health Association, Washington DC.

ANDERSON, J.B., and ZWEIG, H.P., 1962; Biology of Waste Stabilisation Ponds, *Southwest Water Water Works Journal*, **44**, 15-18.

ANSARI, M.Y., 1973; Stabilisation Ponds, *Pollution Abstracts*, **4**, 106.

ARCEIVALA, S.J., 1970; *Waste Stabilisation Ponds Design, Construction & Operation in India*; Central Public Health Engineering Research Institute, Nagpur, India.

ARCEIVALA, S. J., 1972; *Low Cost Wastewater Treatment*; Central Public Health Engineering Research Institute, Nagpur, India.

ARCEIVALA, S.J., 1973; Cost Estimates for Various Sewage Treatment Process of India, *Pollution Abstracts*, **4**, 109.

ARTHUR, J.P.,1981; *The Development of Design Equations for the Facultative Waste Stabilisation Ponds in Semi-Arid Areas*; Institution of Civil Engineering, Washington DC.

ARTHUR, J.P.,1983; *Notes on the Design and Operation of Waste Stabilisation Ponds in Warm Climates*; World Bank Technical Paper No 7, Washington DC.

ASIAN INSTITUTE OF TECHNOLOGY, 1981; *Waste Stabilisation Pond Designs for Tropical Zone*; Department of Environmental Engineering, Asian Institute of Technology, Bangkok.

ASSENZO, J.R., and REID, G.W., 1966; Removing Nitrogen and Phosphorus by Bio-Oxidation Ponds in Central Oklahoma, *Water and Sewage Works*, **13**, 294-299.

AUSTRALIAN INTERNATIONAL ASSISSTANCE BUREAU, 1989; *Annual Report of Australian Aid Projects to Thailand;* Australian International Assisstance Bureau, Bangkok Office, Bangkok.

AVELALLEMANT, S.P., and HELD, J.W., 1980; *Assessment of Sewage Lagoons as Potential Fish Culture Sites in West Central Wisconsin*; A Technical Report Number WIS WRC 80-10, Water Resource Center, Wisconsin.

BACHMANN, G., 1980; Waste Stabilation Ponds and Their Impact on the Application of Appropriate Technology, in *Waste Stabilisation Ponds Design and Operation*, 97-134; A Report of a Seminar at Lahore, 29 December 1979-5 January 1980, World Health Organisation, Geneva.

BANERJI, S.K., and RUESS, B., 1987; Evaluation of Waste Stabilisation Pond Performance in Missouri and Kansas, USA, *Water Science and Technology*, **19**, 39-46.

BANGKOK METROPOLITAN OFFICE, 1987; *A Preliminary Consideration of an Applying Lagoons for Treating Sewage in Bangkok*; A Report Submitted to Ministry of Interior, Bangkok Metropolitan Office, Bangkok.

BARNES, D., and WILSON, E., 1978; *Chemistry and Unit Operations in Sewage Treatment*; Applied Science Publishers Ltd, London.

BARSOM, G., 1973; *Lagoon Performance and the State of Lagoon Technology*; U.S. Environmental Protection Agency, Publication Number EPA-R2-73-144, Washington DC.

BARTONE, C.R., and ARLOSOLOFF, S., 1987; Irrigation Reuse of Pond Effluents in Developing Countries, *Water Science and Technology*, **19**, 289-297.

BARTSCH, A.F., and ALLUM, M.O., 1957; Biological Factors in Treatment of Raw Sewage in Artificial Pond, *Limnology and Oceanography,* **2**, 77-84.

BAUMANN, W., and KARPE, H J., 1980; *Wastewater Treatment and Excreta Disposal in Developing Countries*; German Agency for International Technical Cooperation (GTZ), Eschborn, Germany.

BEADLE, L.D., and HARMSTON, F.C., 1958; Mosquitoes in Sewage Stabilisation Ponds in the Dakotas, *Mosquito News*, **18**, 239-296.

BERNHARD, C., and KIRCHGESSNER, N., 1987; Civil Engineer's Point of View on Water Tightness and Clogging of Waste Stabilisation Ponds, *Water Science and Technology*, **19**, 365-367.

BERSCHAUER, W.A., 1961; Sewage Lagoons with a Difference, *The American City*, **76**, 88-90.

BHASKARAN, T.R., and CHAKRABARTY, R.N., 1966; Pilot Plant for Treatment of Cane Sugar Waste, *Journal of Water Pollution Control Federation,* **38**, 1160-1169.

BOATRIGHT, D.T., and LAWRENCE, C.H., 1977; Rapid Sand Filtration for Best Practical Treatment of Domestic Waste Water Stabilisation Pond Effluent, *Journal of Water Pollution Control Federation*, **49**, 575-583.

BOKIL, S.D., and AGRAWAL, G.D., 1977; Stratification in Laboratory Simulations of Shallow Stabilisation Ponds, *Water Research*, **11**, 1025-1030.

BOUTIN, P., VACHON, A , and RACAULT, Y., 1987; Waste Stabilisation Ponds in France: An Overall Review, *Water Science and Technology*, **19**, 25-31.

BRADLEY, R.M.,1983; BOD Removal Efficiencies in Two Stabilisation Lagoons in Series in Malaysia, *Water Pollution Control*, **82**, 114-122.

BREDLEY, R.M ,and SILVA, M.O.S., 1977; Stabilisation Lagoons Including Experience in Brazil, *Effluent and Water Treatment Journal*, **17**, 21-22.

BRIDLE, T., 1991; Proven and Efficient Processes for the Control and Disposal of Sludge; in JONES, A.J., O'GALLAGHER, B., and HUBICK, K.T. (eds), *Waste Technology and Management*, 37-45; Department of Industry, Technology and Commerce, ACT, Australia.

BRINCK, C.W., 1961; Operating Experiences in Montana: Waste Stabilisation Lagoons, in *Waste Stabilisation Pond*, 112-115; U.S. Public Health Service Publication Number 872, Washington DC.

BUCHBERGER, S.G., 1989; Design of Wastewater Storage Ponds at Land Treatment Sites II: Equilibrium Storage Performance Functions, *Journal of Environmental Engineering*, **115**, 659.

BUCKSTEEG, K., 1987; German Experiences with Sewage Treatment Ponds, *Water Science and Technology*, **19**, 17-23.

BUSH, A.F., ISHERWOOD, J.D., and RODGI, S., 1961; Dissolved Solids Removal from Wastewater by Algae, *Journal of Sanitary Engineering Division*, **87**, 39-57.

CALDWELL, D.H., 1946; Sewage Oxidation Ponds- Performance, Operation and Design, *Journal of Sewage Works*, **18**, 433-458.

CALLOWAY, T., and WAGNER, B., 1966; *Sewage and Lagoons for Developing Countries*; Department of Housing and Urban Development Publication No. 62, Washington DC.

CANTER, L.W., ENGLANDE, A.J., and MAULDIN, A.F.JR., 1969; Loading Rates on Waste Stabilisation Ponds, *Journal of Sanitary Engineering Division*, **95**, 1117-1129.

CANTER, L.W., 1969; *Waste Stabilisation Pond Performance and Effectiveness in the Removal of Pathogenic Organisms*, Tulane University, New Orleans.

CANTER, L.W., and ENLANDE, A.J.JR., 1970; States' Design Criteria for Waste Stabilisation Ponds, *Journal of Water Pollution Control Federation*, **42**, 1846-1847.

CANTER, L.W., MANILA, J.F., GEORGE, K.C.,and LEWIS, S., 1982; Wastewater Disposal and Treatment; in REID, G.W.(ed), *Appropriate Methods of Treating Water and Wastewater in Developing Countries*, 207-270; Bureau of Water and Environmental Resources Research, The University of Oklahoma at Norman, USA.

CHAFFIN, C.M., 1976; Waste Stabilisation Ponds at Texas Eastern Company, in GLOYNA, E.F., MALINA, J.F., and DAVIS, D.M. (eds), *Ponds As a Wastewater Treatment Alternative*; Centre for Research in Water Resources, 205-220; College of Engineering, The University of Texas at Austin, Texas.

CHAUDHURI, M., 1973; Virus Removal in Waste Stabilisation Ponds, *Indian Journal of Environmental Health*, **16**, 20-25.

CLARK, C.E., and KALDA, D.C., 1961; Waste Stabilisation Ponds in South Dakota; in *Waste Stabilisation Lagoons*, 118-123; U.S. Public Health Publication Service Number 872, Washington DC.

CLARK, S.E, 1970; Alaska Sewage Lagoons; in McKINNEY, R.E. (ed), *Proceedings of The International Symposium on Waste Treatment Lagoons*, 221-275; U.S. Department of Interior, Kansas City, Missouri.

COOLEY, C.C., and JENNINGS, R.R., 1961; Virginia's Experimental Installations; in *Waste Stabilisation Lagoons*, 41-52; U.S. Public Health Publication No. 872, Washington DC.

COSSER, P.R., 1982; Lagoon Algae and the BOD Test Effluent and Water Treatment, *Journal of Water Pollution Control Federation*, **22**, 354-358.

CULP, R.L., 1961; Waste Stabilisation Ponds in Kansas; in *Waste Stabilisation Lagoons*, 136-139; U.S. Public Health Service No. 872, Washington DC.

DART, R.K.and STRETTON, R.J., 1987; *Microbiological Aspects of Pollution Control*; Elsevier Publishing Company, London.

DeBUSK, T.A., REDDY, K.R., and CLOUGH, K.S., 1989; Effectiveness of Mechanical Aeration in Floating Aquatic Macrophyte-Based Wastewater Treatment System, *Journal of Environmental Quality*, **18**, 349-354.

De la O, E.E., and MATINEZ, J.A., 1976; Evaluation of Waste Stabilisation Pond Performance in Mexico, in GLOYNA, E.F., MALINA, J.F., and DAVIS, D.M. (eds), *Ponds As a Wastewater Treatment Alternative*; Centre for Research in Water Resources, 274-324; College of Engineering, The University of Texas at Austin, Texas.

DEPARTMENT OF ENVIRONMENT HOUSING AND COMMUNITY DEVELOPMENT, 1975; *Algae Harvesting from Sewage*; Bureau of Environmental Studies, Canberra.

DEPARTMENT OF HEALTH, 1985; *A Report on Environmental Health Problems of the Phong River*; Department of Health, Ministry of Public Health, Bangkok.

DEPARTMENT OF HEALTH, 1991; *Epidemiological Data and Health Status of Thailand in 1990*; Department of Health, Ministry of Public Health, Bangkok.

DEPARTMENT OF INDUSTRY, TECHNOLOGY AND COMMERCE, 1990; *Australia Getting Down to Business*; Department of Industry, Technology and Commerce, ACT.

DEPARTMENT OF POLLUTION CONTROL,1992; *A Summary of Environmental Pollution Acts of Thailand;* Department of Pollution Control, Ministry of Science, Technology and Environment, Bangkok.

DEPARTMENT OF POLLUTION CONTROL, 1992a; *Thailand Environmental Situations Review*; Department of Pollution Control, Ministry of Science, Technology and Environment, Bangkok.

DEPARTMENT OF POLLUTION CONTROL, 1992b; *The National Pollution Control Act 1992*; Department of Pollution Control, Ministry of Science, Technology and Environment, Bangkok.

DEPARTMENT OF POLLUTION CONTROL, 1993; *A Report of Sewage Treatment Systems Studied in Thailand*; Department of Pollution Control, Ministry of Science, Technology and Environment, Bangkok.

DINGES, R.,1982; *Natural System for Water Pollution Control*; Van Nostrand Reinhold Company, New York.

DORNBURSH, J.N., 1976; *Infiltration Lagoons for Tertiary Treatment of Stabilisation Pond Effluent*; South Dakota Water Resources Research Institute, Department of Civil Engineering, South Dakota State University, USA.

ECKENFELDER, W.W., 1970; *Water Quality Engineering for Practising Engineers*; Barnes & Noble, Inc., New York.

ECKLEY, L.E., CANTER, L.W., and REID, G., 1974; *Operation of Stabilisation Ponds in Tropical Areas*; U.S. Army Medical Research and Development Command, Washington DC.

EDWARDS, P., and SINCHUMPASAK, O., 1981; The Harvest of Microalgae from the Effluent of a Sewage Fed High Rate Stabilisation Pond by *Tilapia Nilotica*, *Aquaculture*, **23**, 83-105.

ELLIS, K.V., 1981; Stabilisation Ponds: Effects of Climate, Design Criteria, Land Requirement; in *Waste Stabilisation Ponds Design and Operation*, A Report of a Seminar at Lahore, 155-180; 29 December 1979-5 January 1980, World Health Organisation, Geneva.

ELLIS, K.V., 1983; Stabilisation Ponds: Design and Operation, *Journal of Critical Reviews in Pollution Control*; **13**, 69-102.

ESEN, I.I , 1987; Algal-Bacterial Ponding Systems for Municipal Wastewater Treatment in Arid Regions, *Water Science and Technology*, **19**, 341-343.

FALL, E.B.JR., 1971; Redesigning Existing Facilities to Increase Hydraulic and Organic Loading, *Journal of Water Pollution Control Federation*, **43**, 1695.

FARRELL, J.B., 1984; Pathogen Reduction Studies in EPA'S Sludge Management Program; in *Proceedings of a Conference on Disinfection of Wastewater Effluents and Sludges: State of the Art and Research Needs*, 153-166; May 7-9, 1984, Miami, Florida.

FILIPI, T.A., 1961; Operating Experiences in Nebraska; in *Waste Stabilisation Lagoons*, 144-146; U.S. Public Health Service Publication Number 872, Washington DC.

FISHER, C. P, 1968; Waste Stabilisation Pond Practices in Canada, in GLOYNA, E.F., and ECKENFELDER, W.W. (eds), *Advances in Water Quality Improvement*, 435-449; University of Texas Press, Texas.

FLEMING, J.R., 1962; Tennessee Policy on Sewage Lagoons, *Public Works*, **93**, 199-200.

FLOAN, D,L., 1961; Installing a Sewage Lagoon and Lift Stations, *Water and Sewage Works*, **180**, 194-197.

FRANZMATHES, J.R., 1970; Bacteria and Lagoons, *Water and Sewage Works,* **117**, 90-92.

GAKSTATTER, J.H., 1978; A Survey of Phosphorus Levels in Treated Municipal Wastewater, *Journal of Water Pollution Control Federation*, **50**, 718-722.

GEORGE, D.B., 1982; Lagoons and Oxidation Ponds, *Journal of Water Pollution Control Federation*, **54**, 610-612.

GERARDI, M.H., and GRIMME, J.K., 1987; Insects Associated with Wastewater Treatment: Their Role and Control, *Water Pollution Control Association of Pennsylvania Magazine*, **20**, 19-22.

GLOYNA, E.F., and HERMAN, E.R., 1956; Some Design Consideration for Oxidation Ponds, *Journal of Water Pollution Control Federation*, **82**, 197-229.

GLOYNA, E.F., 1968; Basis for Stabilisation Ponds Designs; in GLOYNA, E.F., and ECKENFELDER, W.W. (eds), *Advances in Water Quality Improvement*, 297-408; University of Texas Press, Texas, Texas.

GLOYNA, E.F. and AQUIRRE, J.,1970; New Experimental Pond Data; in McKINNEY, R.E. (ed), *Proceedings of The International Symposium on Waste Treatment Lagoons*, 200-231; U.S. Department of Interior, Kansas City, Missouri.

GLOYNA, E.F., 1971; *Waste Stabilisation Ponds*; World Health Organisation Monograph Series No. 60, Geneva.

GLOYNA, E.F., 1976; Facultative Waste Stabilisation Pond Design, in GLOYNA, E.F., MALINA, J.F.JR., and DAVIS, E.M. (eds), *Ponds as a Wastewater Treatment Alternative*, 7-23; Centre for Research in Water Resources, College of Engineering, The University of Texas at Austin, Texas.

GOBBIE, M.R., 1990; *A Review of Sludge Treatment and Disposal Techniques*; Australian Water Resources Council, Sydney.

GREWIS, O.E., and BURKETT, C.A., 1966; Two-Thousand Towns Treat Twenty-Thousand Waste, *Journal of Water and Wastes Engeering*, **3**, 54-57.

HALL, D.H., and SHELTON, J.E., 1983; *Advanced Biological Treatment of Municipal Wastewater Through Aquaculture*; A Report to Oklahoma State Department of Health, Oklahoma.

HARRIS, S.E., 1977; Intermittent Sand Filtration for Upgrading Waste Stabilisation Pond Effluents, *Journal Water Pollution Control Federation*, **49**, 83-102.

HARVEY, R.M., and, FOX, J.L., 1973; Nutrient Removal Using *Lemna Minor*, *Journal of Water Pollution Control Federation*, **45**, 1928-1938.

HENDERSON, S., 1978; An Evaluation of the Filter Fishes, Silver and Bighead Carp for Water Quality Improvement, in SMITHERMAN, R.O., SHELTON, W.L., and GROVER, J.H., (eds), *Proceeding of Culture of Exotic Fishes*, 121-126; Fish Culture Section, American Fisheries Society, Auburn, Alabama.

HENRY, J.G., and PRASAD, D., 1985; Microbial Aspects of the Inuvik Sewage Lagoon; *Water Science and Technology*, **19**, 1097-1099.

HERBERT, C.C., NEEL, J.K., and MONDAY, C.A. JR., 1961; *Studies of Raw Sewage Lagoons at Fayette, Missouri*; U.S. Public Health Service Number 875, Washington DC.

HERMAN, E.R., and GLOYNA, E.F., 1958; Waste Stabilisation Ponds-I Experimental Investigations, *Sewage and Industrial Wastes*, **30**, 511-538.

HERMAN, E.R.,1962; Stabilisation Pond as a Nitrate Reducing Reactor; *Journal of the Sanitary Engineering Division*, **88**, 1-20.

HESS, M.L., 1983; *Manual of Design and Operation of Waste Stabilisation Ponds for Use in Hot Regions*; Second Draft, World Health Organisation (WHO-Eastern Mediterranean Regional Office), Alexandria.

HETTIARATCHI, J.P.A, and SMITH, D.W., 1989; Lagoons and Ponds, *Journal of Water Pollution Control Federation*, **61**, 810-814.

HILL, D.O., and SHINDALA, A., 1977; *Performance Evaluation of Kilmichael Lagoon*; College of Engineering, Mississippi State University, Mississippi.

HODGSON, A.P., 1964; Stabilisation Ponds for a Small African Urban Area, *Journal of Water Pollution Control Federation*, **36**, 51-67.

HORNING, W.B., 1964; *Waste Stabilisation Pond Study, Lebanon, Ohio,* U.S. Public Health Service Publication Number 999, Washington DC.

JOHNS, M., 1993; Design Procedures for Facultative Ponds, in *Lagoon Treatment of Municipal Sewage Seminar,* 20-21 August 1993, Australian Water and Wastewater Association, Tasmania.

JOHNSTON, J.E.,1961; Lagoon Development and Acceptance in Mississippi, *in Waste Stabilisation Lagoons*, 89-94; U.S. Public Health Service Publication Number 872, Washington DC.

KABLER, P.W., 1959; Removal of Pathogenic Microorganisms by Sewage Treatment Processes, *Sewage and Industrial Wastes*, **31**, 1373-1382.

KAWAI, H., 1987; Use of Agal-Microcrustracean Polyculture System for Domestic Wastewater Treatment; *Water Science and Technology*, **19**, 65-70.

KHON KAEN CITY COUNCIL, 1987; *A Report on Wastewater Treatment Palnt: A Finacial Management*; Khon Kaen City Council, A Report, Submitted to The Ministry of Interior, Khon Kaen, Thailand.

KHON KAEN MUNICIPALITY, 1986; *The Sewage Lagoon Project*; Khon Kaen Municipality Office, Khon Kaen, Thailand.

KHON KAEN MUNICIPALITY, 1991; *Khon Kaen Municipality Annual Report 1991*; Khon Khan Municipality Office, Khon Kaen, Thailand.

KHON KAEN MUNICIPALITY, 1992a; *Our City Development Schemes for Your Better Living*; Publication Service Number 9, Khon Kaen Municipality Office, Khon Kaen, Thailand.

KHON KAEN MUNICIPALITY, 1992b; *Annual Report of Khon Kaen Municipality*; Khon Kaen Municipality Office, Khon Kaen, Thailand.

KHON KAEN MUNICIPALITY, 1993; *Annual Report of Khon Kaen Municipality*; Khon Kaen Municipality Office, Khon Kaen, Thailand.

KHON KAEN PROVINCIAL OFFICE, 1992; *Khon Kaen Demographical Data 1992*; Khon Kaen Provincial Office, Khoan Kaen Thailand.

KHON KAEN UNIVERSITY, 1986; *A Report of Khon Kaen Wastewater Qualities Study*; A Report Submitted to Thailand Institute of Scientific and Technological Reseach (TISTR) by Department of Environmental Engineering, Khon Kaen University, Khon Kaen, Thailand.

KITTERLE, R.A., and ENNS, W.R., 1968; Aquatic Insects Associated with Midwestern Waste Stabilisation Lagoons, *Journal of Water Pollution Control Federation*, **40**, 41.

KLOCK, J.W., 1972; Sequential Processing in Wastewater Lagoons; *Journal of Water Pollution Control Federation*, **44**, 241.

KLOCK, J.W., 1971; Survival of Coliform Bacteria in Wastewater Treatment Lagoons, *Journal of Water Pollution Control Federation*, **34**, 2071.

LAKSHMINARAYANA, J.S.S., 1972; Performance of Stabilisation Ponds at Bhandewadi, Nagpur; *Pollution Abstracts*, **4**, 107.

LAKSHMINARAYANA, J.S.S., and ABDULAPPA, M.K., 1973; The Effect of Sewage Stabilisation Ponds on Helminths, *Pollution Abstracts*, **4**, 110.

LANSDELL, M., 1987; Development of Lagoons in Venezuela, *Water Science and Technology*, **19**, 55-60.

LIN, P.W., 1981; Algae in Wastewater Treatment and Upgrading of Stabilisation Ponds Effluent; in *Waste Stabilisation Ponds Design and Operation*, 252-266; A Report of a Seminar at Lahore, 29 December 1979-5 January 1980, World Health Organisation, Geneva.

LUDWIG, H.F.,1987; Critically Needed Studies for Guiding Water and Wastewater Technology in Developing Countries; in PANSAWAD, T. (ed), *Proceedings on National Seminar, Water and Wastewater Technology*, 4-8; 12-13 March 1987, College of Engineering, Chulalongkorn University, Bangkok.

LUDWIG, H.F., and BROWDER, G., 1992; Appropriate Water Supply and Sanitation Technology for Developing Countries in Tropical Monsoon Climates, *The Environmentalist*, **12**, 131-139.

LUMBERS, J.P., and ANDOH, R.Y.G., 1987; Identification of Benthic Feed-Back in Facultative Ponds, *Water Science and Technology*, **19**, 177-182.

LYMAN, E.D., 1970; A Field Study of the Performance of Waste Stabilisation Ponds Serving Small Towns; in in McKINNEY, R.E. (ed), *Proceedings of The International Symposium on Waste Treatment Lagoons*, 387-391; U.S. Department of Interior, Kansas City, Missouri.

MACKAY, B.B.Jr, 1978; Municipal Sewage System Springs 20-25 Million Gallon Problem, *Solid Wastes Management*, **21**, 56-58.

MACKENTHUN, K.M., and McNABB, C.D., 1961; Stabilisation Pond Studies in Wisconsin, *Journal Water Pollution Control Federation*, **33**, 1234-1251.

MALINA, J.F.JR., 1973; *Application of Oxygen to Treat Waste from Millitary Field Installations*; A Technical Report on Medical Research and Development Number CRWR-99, Texas University at Austin, Texas.

MALONE, C.D., and SWANN, P., 1978; Make Sewage Works Work; *American City and County*; **93**, 79-81.

MARA, D.D., 1975; Proposed Design for Oxidation Ponds in Hot Climates, *Journal of Environmental Engineering Division*, **101**, 296-300.

MARA, D.D.,1976; *Sewage Treatment in Hot Climates*; ELBS and John Wiley & Sons, London.

MARA, D.D.,1982; Waste Water Treatment in Hot Climates; in FEACHEM, R., McGARRY, M.G. and MARA, D.D. (eds), *Water, Wastes and Health in Hot Climates*, John Wiley & Sons, London.

MARA, D.D., 1988; Waste Stabilisation Ponds: The Production of High Quality Effluents for Crop Irrigation; in *Treatment and Use of Sewage Effluent for Irrigation*, 87-92; Department of Civil Engineering, Leeds University, London.

MARAIS, G.v.R., and SHAW, V.A.,1961; A Rational Theory for the Design of Sewage Stabilisation Ponds in Central and South Africa, *Transactions of the South African Institute of Civil Engineers*, **3**, 205-227.

MARAIS, G. v. R., 1970; Dynamic Behavior of Oxidation Pond; McKINNEY, R.E. (ed), *Proceedings of The International Symposium on Waste Treatment Lagoons*, 15-46; U.S. Department of Interior, Kansas City, Missouri.

MARAIS, G.v.R., 1974; Fecal Bacterial Kinetic in Stabilisation Ponds; *Journal of American Society of Civil Engineers*, **100**, 119-139.

MARTIN, J.D., DUTCHER, V.D., FRIEZE, T.R., TAPP, M., and DAVIS, E.M., 1976; Waste Stabilisation Experiences at Union Carbide Seadrift, Texas Plant; in GLOYNA, E.F., MALINA, J.F., and DAVIS, D.M. (eds), *Ponds As a Wastewater Treatment Alternative*, 190-206; Centre for Research in Water Resources, College of Engineering, The University of Texas at Austin, Texas.

MATHAVAN, G.N., and VIRARAGHAVAN, T., 1991; Waste Treatment Lagoons in Saskatchewan; *International Journal Environmental Studies*, **39**, 237-243.

MAYO, A.W., 1989; Effect of Ponds Depth on Bacterial Mortality Rate; *Journal of Environmental Engineering,* **115**, 964-977.

McGARRY, M. G., 1970; *Water Supply and Wastewater Treatment in Developing Countries*; Asian Institute of Technology, Bangkok.

McGARRY, M.G., and PESCOD, M.B.,1970; Stabilisation Pond Design Criteria for Tropical Asia; McKINNEY, R.E. (ed), *Proceedings of the International Symposium on Waste Treatment Lagoons*, 114-132; U.S. Department of Interior, Kansas City, Missouri.

McGARRY, M.G., 1982; Oxidation Ponds and Fish Culture; in *Water Supply and Sanitation in Developing Countries*, 201-218; Ann Arbor Science Publishing, Missouri.

McGHEE, T.J., and PATTERSON, R.K.,1974; Upflow Filtration Improves Oxidation Pond Effluent, *Water and Sewage Works*, **121**, 82.

McGRIFF, E.C., 1981; *Facultative Lagoon Effluent Polishing Using Phase Isolation*; Environmental Protection Agency Project Report Number EPA-600/S2-81-084, Washington DC.

McKINNEY, R.E., 1976; Functional Characteristics Unique to Ponds; in GLOYNA, E.F., MALINA, J.F.and DAVIS, E.M. (eds), *Ponds As a Wastewater Treatment Alternative*; 283-324; Centre for Research in Water Resources, College of Engineering, The University of Texas at Austin, Texas.

McKINNEY, R.E., 1990; Performance Evaluation of an Existing Lagoon System at Eudora, Kansas; *National Technical Information Service Abstracts Database on Municipal Sewage Treatment Lagoons (ponds)*, April 1977-1990, 61; U.S. Department of Commerce, Washington DC.

MEENAGHAN, G.F., and ALLEY, F.C., 1963; Evaluation of Waste Stabilisation Pond Performance; in *Proceedings of the Third Annual Industrial Water and Waste Conference,* 3-19; Texas Water Pollution Control Association, Rice University, Texas.

MERON, A., REBHUN, M., and SLESS, B.,1965; Quality Changes as a Function of Detention Time in Wastewater Stabilisation Ponds, *Journal of Water Pollution Control Federation* , **37**,1675-1670.

METCALF, L., and EDDY, H.P., 1972; *Wastewater Engineering: Collection Treatment Disposal*; McGraw-Hill Book Company, New York.

METCALF & EDDY INC., 1979; *Wastewater Engineering: Treatment Disposal and Reuse*; McGaw-Hill Book Company, Boston.

METZLER, C.F., and CULP, R.L., 1959; Sewage Ponds Getting Popular, *Engineering News Record*, **162**, 26.

MIDDLEBROOKS, E.J., JONES, N.B., REYNOLDS, J.H., TORPY, M.F., and BISHOP, R.P., 1979; *Lagoon Information Source Book*; Ann Arbor Science Publishers Inc, Michigan.

MILLS, D.A., 1961; Depth and Loading Rates of Oxidation Ponds; *Water and Sewage Works*, **180**, 345-346.

MINISTRY OF FINANCE, 1993; *National Environmental Fund*; Office of Regional Finance, Ministry of Finance, Bangkok.

MINISTRY OF INTERIOR, 1983; *The Study of Thai Regional City Sewage Treatment Alternative*; United Nation Development Programme Project No THA/80/014, Ministry of Interior, Bangkok.

MINISTRY OF INTERIOR,1984; *Survey of Wastewater Sources Chiang Mai and Khon Kaen;* Technical Paper No. 2, Office of Regional Cities Development, Ministry of Interior, Bangkok.

MINISTRY OF INTERIOR, 1985a; *National Sewage Treatment Scheme Funding;* Office of Regional Cities Development, Ministry of Interior, Bangkok.

MINISTRY OF INTERIOR, 1985b; *Design Criteria and Procedures for Wastewater Khon Kaen*; Regional Cities Development Project, Phase III, Technical Paper No.13, AIDAB Project No. 87614, Bangkok.

MINISTRY OF INTERIOR, 1986; *A Revise on Sewage Treatment Programme Fund;* Office of Regional Cities Development, Ministry of Interior, Bangkok.

MINISTRY OF INTERIOR, 1987; *Regional City Sewage Treatment Project Annual Report*; Office of Regional Cities Development, Ministry of Interior, Bangkok.

MINISTRY OF INTERIOR,1989; *The Regional Cities Sewage Treatment Programme: An Advancement Report*; Office of Regional Cities Development, Ministry of Interior, Bangkok.

MINISTRY OF INTERIOR, 1990; *Regional Cities Wastewater Treatment Programmes: A Management Perspective*; Office of Regional Cities Development, Ministry of Interior, Bangkok.

MINISTRY OF INTERIOR, 1991; *Pollution Tax: A Case Study*; Office of Regional Cities Development, Ministry of Interior, Bangkok.

MINISTRY OF PUBLIC HEALTH, 1992; *Khon Kaen Waste Water Treatment*; Department of Health, Bangkok.

MINISTRY OF PUBLIC HEALTH, 1993; *Health Statisics*; Office of Permanent Secretary, Ministry of Public Health, Bangkok.

MITCHELL, B.D., and WILLIAMS, W.D., 1982; *The Performance of Tertiary Ponds and the Role of Algae, Macrophytes and Zooplankton in the Waste Treatment Process*; Department of National Development and Energy, Canberra.

MOELLER, J.R., and CALKINS, J., 1980; Bacterial Agents in Wastewater Lagoons and Lagoon Design; *Journal of Water Pollution Control Federation*, **52**, 2442-2451.

NEEL, J.K, McDERMOTT, J.H., and MONDAY, C.A.JR., 1961; Experimental Lagooning of Raw Sewage at Fayette, Missouri; *Journal of Water Pollution Control Federation,* **33**, 603-641.

NOVAK, S.M., 1976; Biological Waste Stabilisation Ponds at Exxon Company, U.S.A. Baytown Refinery and Exxon Chemical Company U.S.A. Chemical Plant, in GLOYNA, E.F., MALINA, J.F., and DAVIS, D.M. (eds), *Ponds As a Wastewater Treatment Alternative*, 173-189; Centre for Research in Water Resources, College of Engineering, The University of Texas at Austin, Texas.

O' BRIEN, W.J.,1978; Lagoons and Oxidation Ponds Literature Review, *Journal of Water Pollution Control Federation*, **61**, 1093-1096.

ODEGAARD, H., BALMER, P., and HANAEUS, J.,1987; Chemical Precipitation in Highly Loaded Stabilisation Ponds in Cold Climates: Scandinavian Experiences; *Water Science and Technology*, **19**, 71-77.

OFFICE OF NATIONAL ENVIRONMENT BOARD, 1984; *Environmental Qualities of Main Rivers in Thailand*; Ministry of Science, Technology and Energy, Bangkok.

OFFICE OF NATIONAL ENVIRONMENT BOARD, 1985; *A Report of Water Quality Survey in the Chao Phraya River*; Ministry of Science, Technology and Environment, Publication No. 01-288-04, Bangkok.

OFFICE OF NATIONAL ENVIRONMENT BOARD, 1986; *The Waste Stabilisation Pond as City Sewage Treatment Option;* Ministry of Science, Technology and Energy, Bangkok.

OFFICE OF NATIONAL ENVIRONMENT BOARD, 1989; *The National Environmental Quality Standards*; Ministry of Science, Technology and Energy, Bangkok.

OFFICE OF NATIONAL SOCIAL AND ECONOMIC DEVELOPMENT BOARD, 1982; *The Fifth National Social and Economic Development Plan*; Kingdom of Thailand, Office of the Prime Minister, Bangkok.

OFFICE OF NATIONAL SOCIAL AND ECONOMIC DEVELOPMENT BOARD,1986; *The Sixth National Social and Economic Development Plan*; Kingdom of Thailand, Office of the Prime Minister, Bangkok.

OFFICE OF NATIONAL SOCIAL AND ECONOMIC DEVELOPMENT BOARD, 1990; *The Seventh National Social and Economic Development Plan;* Kingdom of Thailand, Office of the Prime Minister, Bangkok.

OFFICE OF NATIONAL SOCIAL AND ECONOMIC DEVELOPMENT BOARD, 1992; *Thailand Pollution Situations*; Office of the Prime Minister, Bangkok.

OFFICE OF THE KING'S AFFAIRS, 1984; *A Report on Monastry Sewage Purification by Using Rotating Biological Contactor System*; Office of the King's Affairs, Bangkok.

O'GALLAGHER, B., 1990; *Waste Management Technologies*; Department of Industry, Technology and Commerce, ACT.

OHKAKI, S., KETRAKANAKUL, A., and PRASERTSOM, U., 1986; Effect of Sunlight on Coliphages in an Oxidation Pond, *Water Science and Technology*, **18**, 37-46.

OKORONKWO, N., and ODEYEMI, O., 1985; Effects of Sewage Effluent on the Water Quality of the Receiving Stream; *Environmental Pollution*, **37**, 71-86.

ORGERON, D.J., 1976; Biological Waste Stabilisation Ponds at E.I. Dupont De Nemours & Company, Serbine River Works, Orange, Texas; in GLOYNA, E.F., MALINA, J.F., and DAVIS, D.M. (eds), *Ponds As a Wastewater Treatment Alternative*, 221-235; Centre for Research in Water Resources, College of Engineering, The University of Texas at Austin, Texas.

ORTH, H.M., and SAPKOTA, D.P., 1988; Upgrading a Facultative Pond by Implanting Water Hyacinth, *Water Reseach*, **22**, 1503-1511.

OSWALD, W.J., 1961; Stabilisation Pond Research and Installation Experiences in California, in *Waste Stabilisation Lagoons*, 33-39; U.S. Public Health Publication Number 872, Washington DC.

OSWALD, W.J., 1963; Fundamental Factors in Stabilisation Pond Design; in ECKENFELDER, W.W., and McCABE, B.J. (eds), *Advances in Biological Waste Treatment*, 357-393; Pergamon Press, Oxford, UK.

OSWALD, W.J., GOLUEKE, C.G., COOPER, R.C., GEE, H.K., and BRONSON, J.C., 1964; Water Reclamation, Algal Production and Methane Fermentation in Waste Ponds, in ECKENFELDER, W.W.(ed), *Advances in Water Pollution Research*, 119-157; The MacMillan Company, New York.

OSWALD, W.J., 1972; Ecological Management of Thermal Discharges, *Pollution Abstracts*, **3**, 101.

OSWALD, W.J., 1976; Experiences with New Pond Designs in California; in GLOYNA, E.F., MALINA, J.F., and DAVIS, D.M. (eds), *Ponds As a Wastewater Treatment Alternative*, 234-272; Centre for Research in Water Resources, College of Engineering, The University of Texas at Austin, Texas.

PARKER, C.D., JONES, H.L., and TAYLOR, W.S.,1959; Purification of Sewage in Lagoons, *Sewage and Industrial Wastes*, **22**, 760-775.

PARKER, C.D., 1962; Microbiological Aspects of Lagoon Treatment, *Journal of Water Pollution Control Federation*, **34**, 149-161.

PARKER, C.D., JONES, H.L, and GREENE, N.G., 1959; Performance of Large Sewage Lagoons at Melbourne, Australia, *Sewage and Industrial Wastes*, **31**, 133-152.

PARKER, C.D.,1970; Experiences with Anaerobic Lagoons in Australia; in McKINNEY, R.E. (ed), *Proceedings of the International Symposium on Waste Treatment Lagoons*, 334-418; U.S. Department of Interior, Kansas City, Missouri.

PARKER, C.E., 1973; *Wastewater Treatment for Small Community*; Virginia Water Resources Research Centre, Virginia University, Virginia.

PEARSON, H.W., 1987; Studies on High Altitude Waste Stabilisation Ponds in Peru, *Water Science and Technology*, **19**, 349-353.

PESCOD, M.B., and MARA, D.D., 1988; Design. Operation and Maintenance of Wastewater Stabilisation Ponds; in *Treatment and Use of Sewage Effluent for Irrigation*, 93-115; Department of Civil Engineering, Leeds University, Leeds.

PIAMPONGSAN, T., 1989; *A Cost-Effective Study of Wastewater Treatment Systems in Suburb Bangkok*; Department of Environmental Promotion, Ministry of Science, Technology and Energy, Bangkok.

PICKFORD, J., 1977; Sewage Treatment in Developing Countries, *Water Pollution Control Federation*, **76**, 65-66.

PIERCE, D.M.,1960; Symposium on Stabilisation Lagoons, *Water and Sewage Works*, **107**, 408-411.

PIERCE, D.M., 1974; Performance of Raw Waste Stabilisation Lagoons in Michigan with Long Period Storage before Discharge; in MIDDLEBROOKS (ed), *Proceedings on Symposium of Waste Stabilisation Largoons*, 89-136; Utah Research Laboratory, Utah.

PIPES, W.O.JR., 1961; Basic Biology of Waste Stabilisation Ponds, *Water and Sewage Works*, **108**, 131-136.

PIPES, W.O.JR., 1962; pH Variation and BOD Removal in Stabilisation Ponds, *Water Pollution Control Federation Journal,* **34**, 1140-1150.

PORGES, R , 1963; Design Criteria for Waste Stabilisation Ponds, *Public Works*, **93**, 99.

PORGES, R., and MACKEHTHUN, K.M., 1963; Waste Stabilisation Ponds: Use, Function and Biota, *Biotechnology and Bioengineering,* **5**, 255-273.

POST, F.J., 1970; Ecology of Selected Bacteria in a Small Intermittent Sewage Pond, *Water Research*, **4**, 341-351.

POTTEN, A.H., 1972; Maturation Ponds: Experiences in their Operation in the United Kingdom as a Tertiary Treatment Process for a High Quality Sewage Effluent, *Water Research*, **6**, 781-795.

PREUL, H.C., and ROESLER, J.F., 1970; Mathematical Simulation of Waste Stabilisation Ponds; in McKINNEY, R.E. (ed), *Proceedings of the International Symposium on Waste Treatment Lagoons*, 395-418; U.S. Department of Interior, Kansas City, Missouri.

PULLEN, K.G., 1973; Experience with Tertiary Treatment at Sewage Works of Lichfield, *Water Pollution Control Federation Journal,* **72**, 52-59.

PURUSHOTHAMAN, K., 1970; Field Studies on Stabilisation Ponds in South India, in McKINNEY, R.E. (ed), *Proceedings of The international Symposium on Waste Treatment Lagoons*, 301-318; U.S. Department of Interior, Kansas City, Missouri.

RADZIEJ, J., 1988; Characteristics of More Important Species of Hydromycrophytes and Their Biomass in a Recultivated Lake Mutek, *Fisheries*, **101**, 103-114.

RAMAN, A., 1972; Studies on Facultative Sewage Lagoons at Kodungaiyu, Madras, *Pollution Abstracts*, **4**, 106.

RAO, N.M., and AGRAWAL, G.D., 1973; Oxidation Ponds to Handle Nitrogenous Fertilizer Factory Effluent, *Pollution Abstracts*, **4**, 107.

RAPP, W.R.JR., 1960; Sewage Lagoon Maintenance, *Journal of Water Pollution Control Federation*, **32**, 660-662.

RAPP, W.R.JR., and EMIL, C., 1965; Mosquito Production in a Eutrophic Sewage Stabilisation Lagoon, *Journal of Water Pollution Control Federation*, **37**, 867-870.

RATTANASUK, S., and GRINSUKON, C., 1981; *Industrial and Community Wastewater Treatment*; Thailand Institute of Scientific and Technological Research, Ministry of Science, Technology and Energy, Bangkok.

REGAN, R.W., and McKINNEY, R.E., 1977; Nitrogen Oxidation and Removal Efficiency Using Activated Algae, *Progress in Water Technology*, **8**, 451-466.

REID, G.W., 1982; Model Acceptance, Lessons of History and Management Concerns, in REID, G.W. (ed), *Appropirate Methods of Treating Water and Wastewater in Developing Countries*, 325-350; Bureau of Water and Environmental Resources Research, The University of Oklahoma at Norman, Oklahoma.

REID, G,W., and MUIGA, M, I.,1982; A Model to Predict Water Demand, Wastewater Disposal, Cost of Treatment Systems, and Equipment Required; in REID, G.W. (ed), *Appropriate Methods of Treating Water and Wastewater in Developing Countries*, 271-324; Bureau of Water and Environmental Resources Research, The University of Oklahoma at Norman, Oklahoma.

REYNOLDS, J.H., 1975; Intermittent Sand Filters for Upgrading Lagoon Effluents, *Public Works*, **106**, 9.

RICHMOND, M.S., 1970; Quality Performance of Waste Stabilisation Lagoons in Michigan, in McKINNEY, R.E. (ed), *Proceedings of the International Symposium on Waste Treatment Lagoons*, 54-63; U.S. Department of Interior, Kansas City, Missouri.

ROGERS, H.G., 1961; Sewage Stabilisation Ponds in Minnesota, in *Waste Stabilisation Lagoons*, 32-35; U.S. Public Health Publication Number 872, Washington DC.

ROSS, S.A., BOIVIN, M.G., and CAVERSON, D.L., 1984; *Operation and Maintenance Costs for Municipal Wastewater Facilities in Canada*; Environmental Protection Service Department, Report No. EPS 5/UP/1, Ottawa.

SAENZ, F.G., 1969; *Sewage Lagoon in Costa Rica*; Latin Americana Technical Cooperation Agency, Costa Rica.

SALEM, S.S., and LUMBERS, J.P., 1987; Initial Evaluation of Sumra Waste Stabilisation Ponds (Jordan), *Water Science and Technology*, **19** , 33-37.

SANKS, R.L., 1977; *Laboratory Studies of Upgrading Effluent Water Quality from Sewage Lagoons*; Research Report No. 82, March 1977, Montana State University, Motana.

SONIASSY, R.N., and LEMON, R., 1986; Lagoon Treatment of Municipal Sewage Effluent in a Subarctic Region Canada, *Water Science and Technology*, **18**, 129-139.

SANTOS, E.J., 1987; High Organic Load Stabilisation Pond Using Water Hyacinth, *Water Science and Technology*, **19**, 25-28.

SANTOS, M.C., and OLIVEIRA, J.F.S., 1987; Nitrogen Transformation and Removal in Waste Stabilisation Ponds in Portugal: Seasonal Variations, *Water Science and Technology*, **19**, 123-130.

SARIKAYA, H.Z., SAATCI, A.M., and ABDULFATTAH, A.F., 1987; Effect of Pond Depth on Bacterial Die-Off, *Journal of Environmental Engineering*, **113**, 1350-1362.

SARIKAYA, H.Z., and SAATCI, A.M., 1988; Optimum Pond Depths for Bacterial Die-Off, *Water Research*, **22** 1047-1054.

SCHURR, K.A., 1970; A Comparison of an Efficient Lagoon System with Other Means of Sewage Disposal in Small Towns; in McKINNEY, R.E. (ed), *Proceedings of the International Symposium on Waste Treatment Lagoons,* 95-100; U.S. Department of Interior, Kansas City, Missouri.

SCHWARTZBROD, J., BOUHOUM, K., and BALEUX, B., 1987; Effects of Lagoon Treatment on Helminth Eggs, *Water Science and Technology*, **19**, 369-371.

SEABROOK, B.L., 1975; *Land Application of Wastewater in Australia; The Werribee Farm System Melbourne, Victoria*; Report No. EPA-430/9-75-017, May 1975, Office of Water Program Operations, Environmental Protection Agency, Washington DC.

SEBASTIAN, S., and NAIR, K.Y.K., 1984; Total Removal of Coliform Domestic Sewage by High-Rate Pond Mass Culture of *Scenedesmus Obliquus*; *Environmental Pollution (Series A)*, **34**, 197-206.

SEIBER, J.N., 1987; *Wastewater Stabilisation Ponds: Principles of Planning and Practice*; World Health Organisation, WHO-Eastern Mediterranean Regional Office, Alexandria.

SHEIKH, M.I., 1981; Waste Stabilation Ponds, Design and Operation; in *Waste Stabilation Ponds, Design and Operation,* 281-293; Proceeding of the Second Working Session, 31 December 1979, A Report of a Seminar at Lahore, 29 December 1979-5 January 1980, World Health Organisation, WHO-Eastern Mediterranean Regional Office, Alexandria.

SHELEF, G., JUANICO, M., and VIKINSKY, M., 1987; Reuse of Stabilisation Pond Effluent for Agricultural Irrigation in Israel, *Water Science and Technology*, **19**, 299-305.

SHILLING, W.C., 1963; Oxidation Ponds- Panacea or Pain, in *Proceedings of the Third Annual Industrial Waste Conference*, 83-87; Texas Water Pollution Control Association, Rice University, Texas.

SIDO, A.D., HARTMAN, R.T., and FUGAZZOTTO, P., 1961; First Domestic Waste Stabilisation Pond in Pennsylviania, *Public Health Reports,* **76**, 201-208.

SILVA, S.A., MARA, D.D., and OVLIVEIRA de R., 1987; The Performance of a Series of Five Deep Waste Stabilisation Ponds in Northeast Brasil, *Water Science Technology*, **19**, 61-64.

SMALLHORST, D.E., 1961; History of Oxidation Ponds in the Southwest, in *Waste Stabilisation Lagoons*, 7-12; U.S. Public Health Publication Number 872, Washington DC.

SOBSEY, M.D., and COOPER, R.C., 1971; Laboratory Studies on the Survival of Poliovirus in Algal-Bacterial Wastewater Treatment Systems, *in Proceedings of the Thirteenth Water Quality Conference*, 137-147; University of Illinois, Department of Civil Engineering, Illinois.

SONIASSY, R.N., and LEMON, R., 1986; Lagoon Treatment of Municipal Sewage Effluent in a Subarctic Region of Canada, *Water Science and Technology*, **18**, 129-139.

STANLER, C.J., and MEIRING, P.G.I., 1965; Employing Oxidation Ponds for Low Cost Sanitation, *Journal of Water Pollution Control Federation*, **37**, 1025-1033.

STEWART, K.M., and ROHLICH, G.A., 1967; *Eutrophication - A Review*; California State Water Quality Control Board, Pubilcation No. 34, California.

STICKNEY, R.R., and HESBY, J.H., 1978; *Tilapia* Production in Ponds Receiving Swine Wastes; in SMITHERMAN, O.R., SHELTON, W.L., and GROVER, J.H. (eds), *Proceedings of the Symposium on Culture of Exotic Fishes*, 90-96; Department of Wildlife and Fisheries Science, Auburn, Alabama.

STEVENS, P.A., 1970; The Role of Oxidation Ponds in Improving Environmental Health in Developing Countries, in McKINNEY, R.E. (ed), *Proceedings of the International Symposium on Waste Treatment Lagoons*, 13-15; U.S. Department of Interior, Kansas City, Missouri.

STEWART, E.A., 1979; Utilisation of Water Hyacinths for Control of Nutrients in Domestic Wastewater; in *Proceedings of Aquaculture Systems for Wastewater Treatment*, 273-293; September 11-12, 1979, University of California-Davis, California.

STOLTENBERG, D.H., 1970; Design, Construction and Maintenance of Waste Stabilisation Lagoons, *Public Works*, **17**, 103-106.

STOUSE, D.C., 1964; Waste Stabilisation Ponds in Iowa-A Survey; *Water Works and Waste Engineering*, **1**, 52-56.

SUPHAPODHOG, A., 1991; Policy and Environmental Management in the Next Decade of Thailand; in SRISATHIT, A. (ed.), *Proceedings of the Conference on Environmental Management in the Next Decade*, 5-8; Environmental Research Institute, Chulalongkorn University, Bangkok.

SVORE, J.H., 1961; History of Sewage Lagoons in the Midwest, in *Waste Stabilisation Lagoons*, 98-112; U.S. Public Health Publication Number 872, Washington DC.

SVORE, J.H., 1964; Sewage Lagoons and Man's Environment, *Civil Engineering*, **34**, 54-56.

TALBOYS, A.P., 1971; Waste Stabilisation Ponds in Latin America, *Journal of Water Pollution Control Federation*, **36**, 51-67.

TAM, D.M., 1982; *Stabilisation Ponds for Sewage Treatment Practical Considerations*; Environmental Engineering Department, Asian Institute of Technology, Bangkok.

TARIQ, M.N., and AHMED, K., 1981; Ecology of Waste Stabilisation Ponds; in *Waste Stabilisation Ponds Design and Operation*, A Report of a Seminar at Lahore, 267-284; 29 December 1979 to 5 January 1980, World Health Organisation, WHO-Eastern Mediteranean Office, Alexandria.

TCHOBANOGLOUS, G., 1991; *Wastewater Engineering: Treatment, Disposal and Reuse*; Third Edition, McGraw-Hill Book Company, New York.

THAILAND DEVELOPMENT AND RESEARCH INSTITUTE, 1987; *Urbanisation and Industrialisation: An Impact to Thailand Environment*; Thailand Development and Research Institute, Asian Institute of Technology, Bangkok.

THAILAND INSTITUTE OF SCIENTIFIC AND TECHNOLOGICAL RESEARCH, 1986a; *A Report on Khon Kaen City Wastewater Analysis: Preparation Phase 1 for Khon Kaen City Wastewater Treatment Project*; Thailand Insititute of Scientific and Technological Research, Ministry of Science, Technology and Energy, Bangkok.

THAILAND INSTITUTE OF SCIENTIFIC AND TECHNOLOGICAL RESEARCH, 1986b; *Report of Khon Kaen Waste Water Treatment Plant*; Thailand Insititute of Scientific and Technology Research, Ministry of Science, Technology and Energy, Bangkok.

THAILAND INSTITUTE OF SCIENTIFIC AND TECHNOLOGICAL RESEARCH, 1987; *The Final Khon Kaen Wastewater Treatment Plan: Ponds Design*; Thailand Insititute of Scientific and Technology Research, Ministry of Science, Technology and Energy, Bangkok.

THAILAND INSTITUTE OF SCIENTIFIC AND TECHNOLOGICAL RESEARCH, 1989; *National Strategies for Wastewater Pollution Control*; Thailand Institute of Scientific and Technological Research, Ministry of Science, Technology and Energy, Bangkok.

THAILAND INSTITUTE OF SCIENTIFIC AND TECHNOLOGICAL RESEARCH, 1993; *The Final Report on Feasibility Study of Sewage Treatment System and Solid Waste Disposal in Samchook and Kao Phra Cities*; Thailand Scientific and Technological Research, Ministry of Science, Technology and Energy, Bangkok.

THAILAND LOCAL GOVERNMENT ASSOCIATION, 1984; *A Possibility of Sewage Treatment Project in Regional Cities*; Thailand Local Government Association Publication, October 1984, Bangkok.

THAILAND LOCAL GOVERNMENT ASSOCIATION, 1986; *City Sewage Treatment Programmes: A Financial Crisis*; Thailand Local Government Association Publication, January 1986, Bangkok.

THAILAND LOCAL GOVERNMENT ASSOCIATION, 1991; *A Revised Country Environmental Standards :How Can We Manage It?*; Thailand Local Government Association Publication, Special Edition, November 1991, Bangkok.

THAILAND LOCAL GOVERNMENT ASSOCIATION, 1993; *A Good Chance for Our City Sewage Projects*; Thailand Local Government Association Publication, October 1993, Bangkok.

THE MERCURY NEWSPAPER, 1993; *Municipal Sewage Improvement Schemes*, 7, 13 July 1993, Hobart, Tasmania.

THEODORE, J., 1956; Sewage Lagoons, *Water and Sewage Works*, **103**, 271-275.

THIRUMURTHI, D.,1974; Design Criteria for Waste Stabilisation Ponds, *Journal of Water Pollution Control Federation*, **46**, 2094.

TOWNE, W.W., and DAVID, W.E., 1957; Sewage Treatment by Raw Sewage Stabilisation Ponds, *Journal of Sanitary Engineering*, **83**, 1337.

TOWNE, W.W., BARTSCH, A.F., and DAVIS, W.H., 1957; Raw Sewage Stabilisation Ponds in Dakota, *Sewage and Industrial Wastes,* **29**, 337-396.

TOWNE, W.W., and HORNING, W.B., 1961; Uses of Stabilisation Ponds in the United States, in *Waste Stabilisation Lagoons*, 68-74; U.S. Public Health Publication Number 872, Washington DC.

URTEAGA, F.C., 1990; The Feasibility of Establishing Operational Water Hyacinth-Based Systems at Treatment Facilities of Existing Cities; *National Technical Information Service Abstracts Database on Municipal Sewage Treatment Lagoons (ponds)*, April 1977-1990, U.S. Department of Commerce, Washington DC.

U.S. ENVIRONMENTAL PROTECTION AGENCY, 1977; *O&M Considerations for Small Municipal Wastewater Treatment Facilities*; Washington DC.

VAN HEUVELEN, W., and SVORE, J.H., 1954; Sewage Lagoons in North Dakota, *Sewage and Industrial Wastes*, **26**, 771-776.

VILLIERS, R.V., and FARRELL, J.B., 1977; Fibre Mats for Landscape Engineering; *Water and Waste Treatment*, **20**, 24.

VIRARAGHAVEN, T., 1973; Removal of Pathogenic Microorganisms by Sewage Treatment Processes, *Pollution Abstracts*, **4**, 110.

VOEGE, F.A., and STANLEY, D.R.,1963; Industrial Waste Stabilisation Ponds in Canada, *Journal of Water Pollution Control Federation*, **35**, 1019-1024.

WALTER, M.V. AND VENNES, J.W., 1985; Occurrence of Multiple-Antibiotic-Resistant Bacteria in Domestic Sewage and Oxidation Lagoons; *Applied and Environmental Microbiology*, **50**, 930-933.

WALTERS, C.F., 1961; Arctic Sewage Lagoons: Waste Stabilisation Lagoons, in *Waste Stabilisation Lagoons*, 89-93; U.S. Public Health Publication No 872, Washington DC.

WEIGAND, R.G., 1983; Comprehensive Program for Surveillance of Package Plants and Stabilisation Ponds, *Journal of Water Pollution Control Federation*, **55**, 1138-1143.

WIDMER, W.J., 1981; Summary Review of Waste Stabilisation Ponds; in *Waste Stabilisation Ponds Design and Operation*, A Report of a Seminar at Lahore, 73-111; 29 December 1979 to 5 January 1980, World Health Organisation, Geneva.

WILLEY, B.R., and BENJES, H.H.JR., 1985; Innovative Water Management Plan Reduced Treatment Costs, *Journal of Water Pollution Control Federation*, **57**, 378-383.

WILLFORD, H.K., and MIDDLEBROOKS, E.J., 1967; Performance of Field-Scale Facultative Wastewater Treatment Lagoon, *Journal of Water Pollution Control Federation*, **39**, 2008-2019.

WILLIAMSON, J.JR., 1956; Sewage Lagoons Are the Answer; *The Consulting Engineer*, **8**, 42-48.

WIXSON, B.G., 1975; *A Study on the Application of Biogrowth Sheets to Improve Lagoon Effluent Quality*; A M.Sc Thesis Missouri University, Missouri.

WOLVERTON, B.C., 1979; Engineering Design Data for Small Vascular Aquatic Plant Wastewater Treatment System; in *Proceedings of Aquaculture Systems for Wastewater Treatment*, 179-192; 11-12 September 1979, University of California-Davis, California.

WORLD HEALTH ORGANISATION, 1973; *Experiencing Waste Stabilisation Ponds from Kenya*; World Health Organisation, Geneva.

WORLD HEALTH ORGANISATION, 1981; *Manual of Waste Stabilisation Pond Operation*; World Health Organisation, Geneva.

WORLD BANK/UNITED NATION DEVELOPMENT PROJECT, 1986; *The Khon Kaen Wastewater Treatment Project Re-Assessment*; United Nation Office of the Asia-Pacific Region, Bangkok.

YANEZ, F.,1980; *Evaluation of the San Juan Stabilisation Ponds - Final Report*; International Development Research Centre/World Health Organisation Publication, Lima, Peru.

YHDEGO, M., 1992; Pilot Waste-Stabilisation Pond in Tanzania, *Journal of Environmental Engineering*, **118**, 286-296.