School of CIVIL Engineering

Modern Design of Waste Stabilization Ponds in Warm Climates: Comparison with Traditional Design Methods

by

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The candidate confirms that the work submitted is his own and that appropriate credit has been given where reference has been made to the work of others.

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ABSTRACT

This dissertation compares traditional and modern methods for designing waste stabilization ponds (WSP) whose final effluent is to be used for the unrestricted irrigation of crops in areas with warm climates in developing countries. Anaerobic and facultative ponds are currently designed based on deterministic average single values of the input design parameters. This design approach is considered as unrealistic and conservative because the input design parameters are subject to uncertainty. Traditional design methods for maturation ponds assume a regime in which the hydraulic flow is completely mixed, and a simple temperature-dependent removal of faecal coliforms.

Modern design methods treat the input design parameters as a range, to take account of their uncertainty; they assume a dispersed hydraulic flow regime when modelling faecal coliform removal in facultative and maturation ponds; and they use different values for faecal coliform removal constant rates in each type of WSP.

Modern methods design the subsequent maturation ponds based on a 95%-ile value of effluent faecal coliform concentration (<1000 FC/100ml) rather than the mean value of effluent faecal coliform concentration, which is the case with traditional design methods. Monte Carlo simulation is used in modern methods, which presents the design outputs across a range of values, which are statistically analysed in percentile form. A Visual Basic computer program has been developed based on Monte Carlo simulations to facilitate the modern design of WSP.

The efficiency of modern and traditional methods of designing WSP are compared in terms of the application of sensitivity analysis, faecal coliform removal, cost of procurement and safety factors. Modern methods are shown to be more flexible than traditional methods and are amenable to sensitivity analysis. Such analysis has found that the concentration of faecal coliforms in effluent is significantly influenced by the per capita BOD, per capita wastewater flow, net evaporation, temperature, faecal coliform removal constant rate in anaerobic ponds, and the dispersion numbers in facultative ponds. Sensitivity analysis has further found that the area of WSP is significantly influenced by the per capita BOD, per capita wastewater flow, population and temperature.

Modern design methods are reliable and produce WSP that are more efficient in removing influent faecal coliform than those designed using traditional methods. Because of this, any exposure of the public to health risks can be realistically assessed when modern methods for design of WSP are used. On the other hand traditional design methods of WSP can result in systems that are inefficient in removing influent faecal coliform due to the underdesign of maturation ponds. Traditional methods typically overestimate the faecal coliform removal constant rate, thereby making it difficult to accurately predict any health risk to which the public might be exposed.

The cost of procuring WSP designed by modern methods is relatively high compared to those when traditional methods are used. The higher costs are attributed to the increased areas of land needed, based upon more realistic estimates of the faecal coliform removal constant rate and the management of the uncertainty of the input design parameters. If WSP designed by traditional methods have a factor of safety of “1” due to the deterministic use of input design parameters, WSP designed by modern methods have a factor of safety of 1.4 and 2.12, based on 50%ile and 95%ile value of WSP area, respectively, assuming that the input design parameters vary by ±20% from their average single design values.

In memory of my father,

Mr. Joseph Kaipa Banda

I know that he would have been truly pleased with this achievement.

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ABBREVIATIONS AND SYMBOLS USED

BOD	biochemical oxygen demand (mg/l)
CO_2	carbon dioxide
d	dispersion number
D	coefficient of longitudinal dispersion (m^2/s)
DNA	Deoxyribonucleic acid
FC	faecal coliform
H	pond depth (m)
H_2S	hydrogen sulphide gas
HS^{-1}	bisulphate ion
K_{FC}	faecal coliform removal constant rate (d^{-1})
K_{BOD}	BOD removal constant rate at temperature (d^{-1})
L	pond length (m)
L_e	effluent BOD (mg/l)
L_i	influent BOD (mg/l)
n	number of subsequent maturation ponds in series
N_i	influent faecal coliform concentration (per 100 ml)
N_e	effluent faecal coliform concentration (per 100 ml)
NH_3	ammoniacal nitrogen
NH_4^+	ammonium ion
Q,	mean design wastewater flow (m^3/day)
r	coefficient of correlation
R^2	coefficient of determination
S^{2-}	sulphide ion
SO_4	sulphate
US	United States
USA	United States of America
υ_i	any random number value (0−1)
W	pond width (m)

WHO	World Health Organization
WSP	waste stabilization ponds
θ	hydraulic retention time (days)
λ_s	surface BOD loading (kg/ha day)
λ_v	volumetric organic loading rate (g/m^3 day)
ϕ	temperature coefficient for faecal coliform removal

NOTE

This pdf version of this MSc(Eng) dissertation is presented on our website as a contribution to knowledge in order to indicate the advantages of designing waste stabilization ponds in warm-climate countries on the basis of parameter-value uncertainty using multi-trial Monte Carlo design simulations.

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CHAPTER 1

INTRODUCTION

1.1 Introduction

This thesis is about the design of waste stabilization ponds (WSP), a low cost technology that is appropriate in the warm climates of developing countries. The thesis compares modern methods for designing WSP with traditional design methods.

1.2 Scope of research

WSP are shallow, rectangular lakes in which domestic and/or industrial wastewater is retained for between 10 and 100 days, depending on the climate, to allow the removal of BOD, excreted pathogens and nutrients. WSP are usually arranged in series of anaerobic, facultative and maturation ponds to improve the efficiency of their performance (Marais, 1974).

Natural biological and physical processes are used to treat the wastewater to the required effluent standard. The quality of the discharged effluent depends on both the process design and the physical design of the WSP. Arthur (1983) and Mara et al. (1992) have reported that poor performance of WSP in developing countries can be attributed to both poor process design and poor physical design.

Each type of WSP carries out a unique function; for this reason the different ponds in the system must be appropriately designed to achieve their intended purpose. Anaerobic and facultative ponds are normally designed for BOD removal, while maturation ponds are designed to remove excreted pathogens.

Numerous process design methods have been proposed by various researchers for anaerobic, facultative and maturation ponds (Hermann and Gloyna, 1958; Gloyna, 1976; Marais and Shaw, 1961; Thirumurthi, 1969; MacGarry and Pescod, 1970; Marais, 1974; Mara, 1976; Arthur, 1983; Mara and Pearson, 1986). These 'traditional' process design methods are grouped into rational design approaches and empirical design approaches (Marecos do Monte and Mara, 1987).

Anaerobic ponds are usually designed by the empirical approach, based on volumetric BOD loading (Mara and Pearson, 1986; Mara et al.1997a). Both rational and empirical design approaches have been used to design facultative ponds. Facultative ponds designed by the rational approach are based on first order BOD kinetics, where the relevant hydraulic flow regime of either plug flow, completely mixed flow or dispersed flow is assumed to reflect the mixing pattern of wastewater pollutants. Facultative ponds are also designed using the empirical design approach based on surface BOD loading. This approach is the one that is usually recommended (Mara, 1987).

Maturation ponds are designed by both rational and empirical approaches, where the removal of excreted pathogens is assumed to follow first order kinetics. The rational design approach assumes that the hydraulic flow regime in the maturation pond is one of complete mixing (Marais, 1974). The first order rate constant for faecal coliform removal has been established by using empirical analysis of observed faecal coliform concentration in existing WSP.

The empirical linear relationship between volumetric BOD loading and pond temperature for designing anaerobic ponds has been developed to avoid anoxic conditions in the pond should the volumetric BOD loading become less than $100 g/m^3/day$. The maximum volumetric BOD loading of $350 g/m^3/day$ has been set to avoid the risk of excessive odour in anaerobic ponds (Mara et al. 1997a). The traditional process design for the volume of an anaerobic pond depends on three variables: flow, wastewater BOD and volumetric BOD loading, which is a function of temperature (Mara and Pearson, 1986).

The traditional process design method for facultative ponds uses the empirical design approach of surface BOD loading before the pond fails (McGarry and Pescod, 1970). Mara (1987) devised a tentative global design equation of surface BOD loading which has incorporated a reasonable factor of safety for stabilizing the algae populations in facultative ponds. Using traditional process design methods, the area of the facultative pond depends on wastewater BOD, design flow, net evaporation, and surface BOD loading, which is a function of temperature.

Traditional process design methods for anaerobic and facultative ponds treat the input parameters as deterministic single average values. In other words, the input parameters are regarded as exact values with 100% certainty. However, Von Sperling (1996) suggests that this approach is unrealistic. Volumetric BOD loading and surface BOD loading are temperature-dependent variables. Temperature varies from a minimum during the cold season to a maximum during the summer. Volumetric BOD loading and surface BOD loading can be manipulated to follow a similar pattern. These parameters should therefore be treated as a range to incorporate any uncertainty.

Wastewater flow is also difficult to design for. The rate of flow varies as the population grows, an issue that is particularly relevant in developing countries. Furthermore, basic design data information is often not readily available in developing countries because of limited resources (Von Sperling, 1996). Industrial wastewater flow and ground water infiltration cannot be measured with high levels of accuracy. These factors can significantly change the wastewater flow from those used in the design. Similarly, wastewater BOD varies as the per capita BOD increases when a community's level of income improves (Campos and Von Sperling, 1996). Also, future improvements of the water supply in developing countries will increase will increase water usage per capita, in turn increasing the volume of domestic wastewater.

These uncertainties call into question the reliability and efficiency of traditional design methods, which treat the input design parameters as deterministic single values and which can result in either the underdesign or overdesign of anaerobic and facultative ponds. A more realistic approach is to consider the input design parameters as a range. Here, the input design parameters are selected randomly within the range that the design engineer can set confidently. Such an approach is realistic and results in the efficient design of WSP, within the limits of the available land and the cost of the project (Von Sperling, 1996).

Currently, the design of maturation ponds is based on calculations of the removal of excreted pathogens (Marais, 1974) where the first order rate constant for faecal coliform removal is temperature dependent. Maturation ponds are efficient in removing excreted pathogens due to their shallowness. This is an important parameter. Curtis and Mara (1994) have observed that high pH, dissolved oxygen concentration, sunlight and high algae concentration are associated with shallow ponds. Shallow ponds allow sunlight to penetrate the pond fully, which facilitates effective photo-oxidation processes that kill excreted pathogens.

The pioneering work on the design of maturation ponds carried out by Marais (1974) has been based on complete mixing due to wind and temperature effects. The first order rate constant model for faecal coliform removal is based on temperature. However, this simple model has faced strong criticism from various researchers (Thirumurthi, 1974; Polprasert et al. 1983; Mayo, 1995; Saqqar and Pescod, 1992; Sarikaya and Saatci, 1987a; Sarikaya et al. 1987b; Qin et al. 1991; Curtis et al. 1992; Von Sperling, 1999).

The completely mixed hydraulic flow regime that Marais assumed in order to develop the faecal coliform removal model is considered unrealistic because short-circuiting and dead spaces cannot be eliminated in WSP (Milton and Harrison, 2003). Faecal coliforms cannot become completely mixed in a WSP, the theoretical hydraulic retention time is impossible to achieve. Marais' suggestion that wind and temperature can result in the complete and instantaneous mixing of a pond's contents is considered unrealistic.

Many researchers have suggested that a dispersed hydraulic flow regime more accurately reflects the conditions of non-ideal flow in WSP (Thirumurthi, 1969; Thirumurthi, 1974; Marecos do Monte and Mara, 1987; Arceivala, 1983; Polprasert and Bhattarai, 1985; Qin et al. 1991; Von Sperling, 1999). Milton and Harrison (2003) and Tchobanoglous at al. (2003) have suggested that non-ideal flow takes into account the short-circuiting which normally occurs in wastewater treatment plants. The first order constant rate for faecal coliform removal based on time and temperature does not reflect the complex environmental conditions prevalent in WSP (Saqqar and Pescod, 1992; Curtis and Mara, 1994).

Various researchers have proposed faecal coliform removal constant rate models assuming a dispersed hydraulic flow regime and complex environmental conditions within the WSP (Polprasert et al. 1983; Sarikaya and Saatci, 1987a; Qin et al. 1991; Saqqar and Pescod, 1992; Curtis and Mara, 1994; Mayo, 1995). However, these models have limitations for designing maturation ponds globally because the environmental variables require certain assumptions to be made. This creates uncertainty about whether the models can accurately predict the rate of removal of excreted pathogens. Furthermore, the use of the equation of Wehner-Wilhelm (1956) requires the dispersion number to be known if a dispersed hydraulic flow regime is to be assumed. In most cases, dispersion numbers have to be predicted from tracer studies carried out in existing WSP. However, when a new WSP is being designed, it is quite possible that there are no others in similar environmental conditions from which to obtain the necessary data.

Von Sperling (1999) proposed empirical equations for designing maturation ponds, with dispersion numbers based on a dispersed hydraulic flow regime. The equations of Von Sperling (1999) predicted accurately the observed faecal coliform removed in 33 ponds in Brazil (R^2 = 0.959). Although these equations for faecal coliform removal and dispersion number are empirically and hydraulically sound, the input design parameters cannot be considered as deterministic values. They are subject to uncertainty because of the changes of environmental conditions that take place in WSP. For this reason, using the equations of Von

Sperling (1999) equations, based on single average values of input design parameters, cannot represent a realistic procedure for the design of maturation ponds.

In summary, traditional methods of process design for WSP are unreliable, resulting in either underdesign or overdesign of the system, with implications for the costs of the project. Unrestricted irrigation of crops with effluent treated by WSP built using traditional process design techniques could pose a health risk to the public if the die-off rate of faecal coliforms has been overestimated by Marais's procedure. The use of average single values of the daily minimum and maximum temperature in the coolest month to establish the safety factor for the design of anaerobic and facultative ponds is not recommended.

This thesis seeks to compare WSP designed by modern methods with those designed by traditional methods. Analysis of the modern methods will show how WSP can be designed to comply with unrestricted crop irrigation guidelines on a 50%ile basis, or 95%ile basis depending on the acceptable health risk and the available project cost. The modern design methods integrate Monte Carlo simulations, the equations of Von Sperling (1999) for faecal coliform removal in facultative and maturation ponds, and the equation of Mara (2002) for faecal coliform removal in anaerobic ponds.

1.3 Research methodology

Modern design methods for WSP require a desktop computer. A Visual Basic programming language was used to write a computer program to design WSP producing treated water suitable for unrestricted crop irrigation. The computer program is run in an Excel spreadsheet so that the design output data that are generated can be analysed statistically in the form of cumulative frequency curves. This program is available from the author upon request.

1.4 Thesis

The aim of this thesis was to compare modern and traditional methods of designing WSP in countries with a warm climate. This was done by:

1. Designing WSP for unrestricted crop irrigation using the equations of Von Sperling (1999) for faecal coliform removal in facultative and maturation ponds, and the equation of Mara (2002) for faecal coliform removal in anaerobic ponds based on a 95-%ile effluent faecal coliform concentration (<1000 faecal coliforms per 100 ml).

2. Developing a computer program in Visual Basic to run in Excel Spreadsheet based on Monte Carlo simulations to manage the uncertainty of the input design parameters.
3. Showing how sensitivity analysis can be carried out to determine those input design parameters that significantly influence the design output of WSP.
4. Comparing the efficiency of faecal coliform removal of WSP designed by modern methods with those designed by the procedures of Marais (1974).
5. Comparing the efficiency of modern design methods with traditional design methods for WSP in terms of land requirements, cost and safety.

This thesis is arranged in the following order:

Chapter 2 is a critical review of the literature on wastewater treatment in WSP, their types and function.

Chapter 3 presents the principles of traditional process design methods that are used for anaerobic, facultative and maturation ponds. It appraises the literature on the current use of deterministic single average values of input design parameters and coefficient values used in design models. The completely mixed hydraulic flow regime that has been adopted in traditional design methods is critically reviewed.

Chapter 4 presents modern methods used in designing WSP. It reviews the justification of adopting a dispersed hydraulic flow regime in modelling the hydraulic performance of WSP. It reviews the literature on the uncertainty of input design parameters and coefficients used in design models. It outlines the principles of Monte Carlo simulations and how these can be used to design WSP in a way that manages the uncertainty of the input design parameters. Empirical design approaches using the equations of Von Sperling (1999) and Mara (2002) are presented for designing maturation ponds for unrestricted crop irrigation.

Chapter 5 outlines the development of a Visual Basic computer program to design anaerobic ponds, facultative pond and maturation ponds by integrating Monte Carlo simulations, the empirical equations of Von Sperling (1999), and the empirical equations of Mara (2002) for faecal coliform removal in WSP. The chapter shows how maturation ponds can be designed based on 95-%ile of effluent faecal coliform concentration (<1000 FC per 100ml).

Chapter 6 outlines the efficiency of modern design methods in carrying out sensitivity analysis on the input design parameters that significantly influence the design output.

Comparisons of the efficiency of faecal coliform removal in WSP designed by modern methods with those based on the equations of Marais (1974) are presented. Finally, the design efficiency of modern methods and traditional methods is compared in terms of land requirement, cost and safety.

Chapter 7 presents the conclusions and recommendations of the thesis.

CHAPTER 2

WASTEWATER TREATMENT IN WSP

2.1 General description of WSP

Mara (1976) describes WSP as large shallow basins enclosed by earthen embankments in which wastewater is biologically treated by natural processes involving pond algae and bacteria. WSP comprise a single series of anaerobic, facultative and maturation ponds or several of such series in parallel. A long hydraulic retention time is necessary because of the slow rate at which the organic waste is oxidized. Typical hydraulic retention times range from 10 days to 100 days depending on the temperature of a particular region.

WSP are considered as the most effective and appropriate method of wastewater treatment in warm climates where sufficient land is available and where the temperature is most favourable for their operation (Mara, 1976). WSP are not only restricted to countries with warm climates; they have also been used in regions with cold climates in Europe and the USA (Abis, 2002). The US Environmental Protection Agency (1983) reports that about 7000 pond systems in USA are employed for treatment of a range of wastewaters, from domestic wastewater to complex industrial wastes.

Anaerobic, facultative and maturation ponds are the three major types of pond in a WSP system. These ponds are normally arranged in series to achieve effective treatment of raw wastewater (Marais, 1974). Anaerobic and facultative ponds are employed for BOD removal, while maturation ponds remove excreted pathogens.

A series of anaerobic and facultative ponds can treat wastewater to a sufficient degree to allow it to be used in a restricted way for irrigating crops. It has been argued that such pond systems remove nematode eggs significantly by sedimentation (WHO, 1989). Maturation ponds are normally used if the treated wastewater is to be used for unrestricted crop irrigation complying with WHO guidelines of less than 1000 faecal coliforms (FC) per 100 ml (WHO, 1989). Maturation ponds have also been used when stronger wastewaters with high concentrations of nutrients (nitrogen, phosphorus) are to be treated prior to surface discharge (Mara, 1997).

2.2 Types of WSP and their functions

2.2.1 Anaerobic ponds

Anaerobic ponds are commonly 2-5m deep and receive a high organic BOD loading (> 100g BOD/m^3/day). They contain no dissolved oxygen and algae. Mara and Pearson (1998) suggest that a thin film of *Chlamydomonas* is occasionally seen at the anaerobic pond surface. The primary function of anaerobic ponds is to remove BOD. The suspended solids settle by gravity to the bottom of pond where they are degradeded anaerobically.

Mara and Pearson (1998) argue that a properly designed anaerobic pond can achieve around 60% BOD removal at 20^{o}C and over 70% at 25^{o}C. Anaerobic ponds operate extremely well with a short retention time and it is not good practice to design anaerobic ponds for long retention times. Anaerobic pond functions satisfactorily when the pond's temperature is above 15^{o}C and the pH is above 6. Under these conditions, it is thought that anaerobic oxidation of settled solids is at a maximum with minimum sludge accumulation. When the pond temperature is less than 15^{o}C, BOD removal is caused by sedimentation with significant loss of the hydraulic retention time (Mara, 1976).

Tchobanoglous et al. (2003) describe how anaerobic heterotrophic bacteria carry out BOD removal. Acid-forming bacteria, (*Streptococcus, Clostridium*, and *Staphylococcus*), methane-forming bacteria (*Methanobacterium* and *Methanospirillum*), and sulphate-reducing bacteria (*Desulfovibrio*) oxidize the organic compounds contained in raw wastewater to obtain food and energy for growth. The acid-forming bacteria initiate redox reactions within organic compounds contained within the waste. These reactions form volatile fatty acids (acetate, propionate, and butyrate). The methane-forming bacteria oxidize the volatile fatty acids products and reduce carbon dioxide to form methane gas. If the raw wastewater contains sulphate compounds (SO_4), the sulphate-reducing bacteria oxidize the organic compounds and reduce the sulphates to form sulphides, including hydrogen sulphide gas (H_2S), the source of odour in anaerobic ponds.

Sawyer et al. (1994) observed that in aqueous solution, hydrogen sulphide is present as either dissolved hydrogen sulphide gas (H_2S), bisulphide ion (HS^{-1}) or sulphide ion (S^{-2}) depending on the pond's pH. A well-designed anaerobic pond with pH of 7.5 favours formation of the less odorous bisulphide ion (Mara and Pearson, 1998). Limiting the concentration of sulphate compounds in raw wastewater to less than 300mg SO_4 per litre is thought to reduce the formation of odorous hydrogen sulphide gas (Gloyna and Espino, 1969). Odour release can be eliminated at the design stage by keeping volumetric BOD loading to less than 350g BOD/m^3/day (Arthur, 1983; Mara and Mills, 1994).

Pescod (1996) and the US Environmental Protection Agency (1983) have suggested operational methods of preventing the release of malodorous compounds by anaerobic ponds. One suggested method is to re-circulate facultative pond effluent to maintain a thin aerobic layer at the surface of the anaerobic pond. This method is considered to prevent the transfer of odours to the air. However, the cost of pumping facultative pond effluent has to be incurred and this can increase the annual operation cost of the system. The second operational method suggested is the addition of lime into the anaerobic pond to increase the pond's pH. This method suppresses the release of odorous gas (H_2S) and favours the formation of non-odorous sulphide compounds. However, this method has also cost implications. The cost of lime can contribute to a substantial increase in the annual operating costs of the WSP.

A small amount of sulphide is known to be beneficial in removing heavy metals found in anaerobic ponds. Mara and Mills (1994) suggest that sulphide reacts with heavy metals to form insoluble metal sulphides that precipitate out. However, Pfeffer (1970) observed that a sulphide concentration of between 50 and 50 mg/l inhibits methanogenesis and this could reduce the performance of the anaerobic pond in terms of BOD removal.

Oragui et al. (1993) observed that a small concentration of sulphide in anaerobic ponds can help remove *Vibrio Cholera*. Curtis and Mara (1994) suggest that environmental conditions in facultative and maturation ponds are not effective in removing *Vibrio Cholera*. Therefore, inclusion of an anaerobic pond in a WSP system can reduce health risks associated with the presence of *Vibrio Cholera* in water that is used for unrestricted crop irrigation.

Inclusion of anaerobic ponds in WSP systems has been noticed to reduce significantly the overall land requirement (Marais, 1970; Arthur, 1983; Pearson et al. 1996a). Mara (1997) showed that inclusion of anaerobic ponds in WSP systems can result in a reduction in land requirements of up to ~50% for systems treating typical domestic wastewater. Designers of WSP in developing countries should therefore include anaerobic ponds in their feasibility studies of WSP systems.

2.2.2 Facultative ponds

Facultative ponds normally follow anaerobic ponds in a WSP system. They are usually 1–2 m deep and are geometrically designed to have a high length-to-width ratio (up to 10:1) to simulate a hydraulic plug flow regime (Mara et al. 1992).

There are two types of facultative pond: primary facultative ponds that receive raw wastewater and secondary facultative ponds that receive settled wastewater effluent from the anaerobic pond. The term facultative is used because both aerobic and anaerobic conditions are found in the pond (Mara and Pearson, 1998). Aerobic conditions are maintained in the upper layers while anaerobic conditions exist towards the bottom of the pond.

Pond depths of less than 1 m are thought to promote the growth of emergent vegetation (Mara, 1976). The shaded environment caused by vegetation is a suitable breeding ground for mosquitoes and snails, both of which may act as vectors of disease (Curtis and Mara, 1994). Depths greater than 1.5 m are avoided to ensure that the bulk of the pond is aerobic. Mara (1976) observed that greater depths (>1.5 m) cause anaerobic conditions and the pond becomes less resistant to shock loads.

The lower anaerobic layer of a facultative pond is dominated by the same anaerobic mechanisms that occur in anaerobic ponds. These conditions are very effective in BOD removal (Marais, 1970).

Facultative ponds are designed for BOD removal based on a surface loading of 100–350 kg BOD/ha day to permit the development and stability of a healthy algae population. This is important because the oxygen for BOD removal used by pond bacteria is mostly supplied by algal photosynthesis (Mara and Pearson, 1998).

2.2.2.1 Algae

Facultative ponds are coloured dark green due to algae. Sometimes facultative ponds appear red or pink when they are overloaded due to the presence of anaerobic purple sulphide oxidizing photosynthetic bacteria (Mara, 1997). Pond algae that predominate in turbid waters of facultative ponds are the motile genera such as *Chlamydomonas, Pyrobotrys, and Euglena* (Mara, 1997). These algae species are capable of optimising their vertical position in the pond water column to incident light intensity and temperature. Non-motile algae such as *Chlorella* are also found in facultative ponds and they rely on mixing initiated by wind and temperature.

The mixing mechanisms enable them to access the incident light, allowing them to photosynthesize. The concentration of algae in a healthy facultative pond depends on surface BOD loading and temperature. Mara (1997) estimated a range of 500 - 2000µg chlorophyll *a* per litre. A concentration of pond algae below this range indicates possible failure of the facultative pond.

The majority of the algae occupy a band about 200mm deep that moves up and down the water column, presumably in response to nutrient concentration and incident light during the day (Pearson et al. 1987a). Anaerobic reactions initiated by acid-forming, methane-forming bacteria and sulphate-reducing bacteria in the lower layers may feed volatile fatty acids and sulphides to the upper zone layer where they may affect the ecology of the pond algae. These products of anaerobic digestion have been observed by Pearson et al. (1987b) to inhibit both growth and production of pond algae. It has been argued by Pearson et al. (1987b) that high ammonia and sulphide concentrations in a facultative pond may result in the replacement of *Euglena* with more tolerant algae species such as *Pyrobotrys* and *Chlorella.*

The algae biomass and its productivity cause a marked diurnal and vertical variation in the levels of dissolved oxygen, pH, sulphide and ammonia. It has been observed that when carbon dioxide is taken up faster than bacterial respiration can supply, the concentration of carbon dioxide drops causing a dissociation of the bicarbonate ion to form carbon dioxide and alkaline hydroxyl (Mara and Pearson, 1998). This raises the pH levels in facultative ponds. Pearson et al. (1987c) observed a rise of pH in facultative ponds exceeding 9.0 and this is important in killing faecal coliforms.

Ammonia and sulphide toxicity have been observed to be pH-dependent (Konig et al. 1987). As the pH of a facultative pond increases, the unionized form of ammonia increases while sulphide production decreases. The effect of this toxicity is to inhibit algae growth and production and these mechanisms are thought to be self-sustaining (Konig et al. 1987).

2.2.2.2 Mixing

Mara (1976) suggests that wind and heat are the two factors that influence the degree of mixing that occurs within facultative and maturation ponds. It is considered that mixing minimizes hydraulic short-circuiting and the formation of stagnant regions in ponds. Mixing has also been considered to distribute uniformly BOD, algae, oxygen and bacteria throughout the water column. It is not possible for WSP to undergo a complete mix as suggested by

Marais (1974). Mixing in WSP attains a certain degree only; the effects of short-circuiting and stagnant regions cannot be eliminated.

In the absence of wind mixing, stratification has been noted to form (Mara, 1976). The less dense upper layer lies on to top of the cooler, denser layer. It has been noted that the two regions are separated by a sharp thermocline (Mara, 1976). Stratification is detrimental to pond performance as it promotes short-circuiting and reduces algal productivity. This agrees with the idea that a complete-mix hydraulic flow model fails to mimic the exact hydraulic flow regime that occurs in WSP.

2.2.2.3 Bacteria

A wide range of heterotrophic bacteria is found in facultative ponds. Mara and Pearson (1998) and the US Environmental Protection Agency (1983) list *Pseudomonas, Archromobacter, Beggiato, Alcaligenes, and Bacillus* as some of the aerobic heterotrophic bacteria that remove BOD in facultative ponds. These aerobic heterotrophic bacteria oxidize organic compounds (electron donor) and reduce oxygen (electron acceptor) supplied by pond algae. The products of aerobic heterotrophic reaction are carbon dioxide (CO_2), ammonium (NH_4^+) and phosphate compounds (PO_4^{2-}). These products are used by algae for growth and production. Aerobic heterotrophic bacteria are thought to have a mutual relationship with algae.

2.2.3 Maturation ponds

Maturation ponds are normally used in series with facultative ponds. They are usually 1–1.5 m deep and are geometrically designed to have a high length-to-width ratio (up to 10:1) to simulate a hydraulic plug flow regime (Mara et al., 1992). The primary function of maturation pond is to remove excreted pathogens to enable the practice of unrestricted crop irrigation (WHO, 1989). The removal of BOD and nutrients has been observed to take place in maturation ponds.

The size and number of maturation ponds depends on the required bacteriological quality of the final effluent. Maturation ponds show less vertical biological and physicochemical stratification (Curtis and Mara, 1994), and they are well oxygenated throughout the day. The algal populations in maturation ponds are more diverse than those in facultative ponds, with non-motile genera tending to be more common (Mara and Pearson, 1998). *Chlorella* has often been observed to be the predominant algal species. It has been argued by Pearson et al.

(1987b) that the predominance of *Chlorella* may be attributed to its resistance to ammonia toxicity. The high pH which is often found in maturation ponds increases the toxicity of ammonia as this favours the formation of the more toxic un-ionized form of ammonia.

Efficient removal of excreted pathogens in maturation ponds depend on the effective and efficient design of the ponds (Mara, 1997). It has been observed by Troussellier et al. (1986) that similar mechanisms for removing excreted pathogens operate in both facultative and maturation ponds and that these mechanisms vary seasonally as they are influenced by environmental conditions.

Many empirical methods for designing maturation ponds use environmental factors, which are thought to influence pathogen removal. As it is technically difficult to count individual pathogenic bacteria and viruses found in wastewater, WSP design and monitoring are done with faecal coliform bacteria, which are used as a pathogen indicator (Feachem et al. 1983).

2.3 BOD removal

Wastewater contains high levels of BOD that need to be removed before the treated effluent can be discharged to receiving water bodies. In anaerobic ponds, BOD removal is achieved by sedimentation of settled solids. The settled organic waste is digested anaerobically, resulting into formation of sludge layer. Because anaerobic ponds contain no dissolved oxygen, Tchobanoglous et al. (2003) argue that oxidation of organic compounds by anaerobic heterotrophic bacteria results in the formation of methane, carbon dioxide and sulphides compounds. Intense anaerobic digestion has been reported to occur at temperatures above 15^{o}C with release of biogas (70% methane, 30% carbon dioxide) (Mara and Pearson, 1998). Anaerobic heterotrophic bacteria are sensitive to a pH of less than 6.2. This requires acidic wastewater to be neutralized prior to treatment in anaerobic ponds. A properly designed anaerobic pond with volumetric BOD loading as recommended by Mara et al. (1997a) can achieve around 60% BOD removal at 20^{o}C and over 70% at 25^{o}C with 1-day retention time.

The settled wastewater that enters the secondary facultative pond is oxidized further by aerobic heterotrophic bacteria. Photosynthesizing algae supply oxygen, which is used by the aerobic heterotrophic bacteria to oxidize the organic wastes. Some oxygen and carbon dioxide has been noted to come from the atmosphere by mass transfer, but the supply of oxygen and carbon dioxide in a facultative pond is thought to be provided mainly by algal-bacterial mutualism (Mara, 1976).

In primary facultative ponds, the functions of both anaerobic and secondary facultative ponds are combined. Marais (1970) observed that 30% of the influent BOD leaves the primary facultative pond in the form of methane gas.

A small amount of BOD removal occurs in maturation ponds due to their lower algal concentrations (Mara, 1997). These lower algal concentrations in the maturation ponds are due to the decreased supply of nutrients and predation by protozoa and micro-invertebrates such as *Daphnia*. It is suggested by Mara (1997) that 70–90% of the BOD of maturation pond effluents is due to algae.

2.4 Pathogen removal

Curtis and Mara (1994), citing Jones (1988), argue the suitability of faecal coliforms as indicators of pathogens in WSP. It is suggested that most faecal coliforms found in WSP are *E.coli.* Studies carried out on the DNA of *E.coli* and some classical pathogens such as shigellae and salmonellae have shown them to have a similar structure, with the notable exception of *Vibrio* spp and *Campylobacter* spp. *Campylobacter* spp. has different DNA structure from *E.coli*, but it has been observed to die more quickly in WSP than faecal coliforms due to its vulnerability to oxygen (Curtis and Mara, 1994).

The case of *Vibrio cholerae* is well documented by Curtis and Mara (1994). It is thought that the environmental conditions in facultative and maturation ponds fail to remove *Vibrio cholerae* because the DNA structures of this pathogen and faecal coliforms are quite different. Research carried out by Oragui et al. (1993) showed that anaerobic ponds are very effective in removing *Vibrio cholerae* due to the formation of toxic sulphide compounds. Anaerobic ponds should be the first choice option when treated effluent is to be reused for unrestricted crop irrigation as these ponds have been shown to be very effective in removing *Vibrio cholerae*.

2.4.1 Bacteria

Mara and Pearson (1998) state that faecal bacteria are removed very efficiently in facultative and maturation ponds. The size and number of maturation ponds determine the numbers of faecal coliforms in the final effluent. Anaerobic ponds play a significant role in removing faecal coliforms through the sedimentation of bacteria associated with solids.

Curtis and Mara (1994) and Curtis et al. (1992) list the principal mechanisms for faecal bacterial removal in facultative and maturation ponds as:

- Time and temperature,
- High pond pH, mainly above 9.4, and
- High light intensity together with a high dissolved oxygen concentration.

Marais (1974) used time and temperature parameters to model faecal coliform removal in WSP. Feachem et al. (1983) observed that faecal coliforms die off in WSP when both time and temperature increase.

High pH values of more than 9.4 have been observed to occur in facultative and maturation ponds due to rapid photosynthesis by pond algae. The high uptake of carbon dioxide by algae causes carbonate and bicarbonate ions to dissociate, resulting in the formation of hydroxyl ions that raises the pond pH to high base values. Faecal bacteria have been observed to die quickly within minutes when the pH exceeds 9.4 (Pearson et al. 1987c; Pahard and Rao, 1974).

Vibrio cholerae has been observed to show resistance to the high pH conditions found in facultative and maturation ponds. Oragui et al. (1993) showed that anaerobic ponds are very effective in removing the pathogens. It is suggested that the small concentrations of sulphide formed by the sulphate-reducing bacteria destroy *Vibrio cholerae*.

Curtis and Mara (1994) and Curtis et al. (1992) observed that high light intensity and high dissolved oxygen concentration are very effective in killing faecal coliforms. It has been argued that light of wavelength 425–700 nm is absorbed by the humic substances ubiquitous in wastewater. It has been suggested that photochemical reactions occur within organic molecules, resulting in electronic excitation which persists for a sufficiently long time to damage faecal coliform cells.

2.4.2 Viruses

The removal mechanism of viruses in WSP has not been researched in detail. Work by Oraqui et al. (1987) showed that removal of viruses in WSP can be attributed to the adsorption of viruses by settleable solids and pond algae with consequent sedimentation. This was observed for rotavirus and enterovirus that were removed in anaerobic ponds. Further removal of viruses is thought to occur in facultative ponds and maturation ponds since rotavirus have been observed to adsorb on to algae (Curtis and Mara, 1994). Curtis and Mara (1994), through personal communication with Oragui, believe that sunlight plays a vital role in removing viruses.

2.4.3 Parasites

The parasites that pose the biggest public health from using effluent from WSP are protozoa and helminths. Shephard (1977) showed that helminth eggs and protozoan cysts are denser than water and so are removed quickly by sedimentation. Mara (1997) reports the settling velocities of protozoa cysts and helminths eggs as high as $3.4x10^{-4}$ m/s. Egg and cyst removal occurs mainly in anaerobic and facultative ponds.

Curtis and Mara (1994) state that *Schistosoma* eggs are able to hatch in facultative and maturation ponds. Anaerobic ponds are considered hostile to *Schistosoma* eggs. It is thought that miracidia can infect snail hosts if they are allowed to colonize the pond. The infectious cercariae can only survive for about 24 hours and are not expected to leave facultative and maturation ponds, which are normally designed to have a minimum hydraulic retention time of three days (Marais, 1974).

2.5 Nutrient removal

2.5.1 Nitrogen

About 60–75% of total nitrogen in raw wastewater is in the form of ammonia, whilst the rest is organic nitrogen (Reed, 1985). Nitrogen cycling is thought to take place in WSP. Organic nitrogen is hydrolysed to ammonia in anaerobic ponds and this tends to increase the ammonia concentration in anaerobic pond effluent (Mara, 1997). Mara and Pearson (1998) suggest that ammonia removal in WSP can be high as 95%, while total nitrogen removal can reach 80%.

Removal of ammoniacal nitrogen in WSP has been well documented (Reed, 1985) and the following pathways suggested:

- Ammonia volatilization (loss of ammonia as a gas to the atmosphere),
- Ammonia assimilation into the algal biomass,
- Physical adsorption and sedimentation, and
- Aerobic nitrification.

2.5.1.1 Ammonia volatilization

Ammonia in water exists in equilibrium between the ionized (NH_4^+) and the free (NH_3) forms. The free NH_3 form is volatile and may be lost to the atmosphere as a gas in a physical process. The equilibrium between the two forms depends on temperature and pH. At a pH above 6.6, NH_4^+ starts to convert to the volatile NH_3 form; at pH 9.2 the two forms exist in equilibrium, and at pH 12, all the ammonia is in the form of NH_3 (Reed, 1985). Konig et al. (1987) observed that a rise of 10 °C in temperature doubles the concentration of NH_3 at a given pH. It is argued that loss of ammonia by volatilization is more likely to occur at higher temperatures and higher pH. High pH conditions (>9.0) are more common in facultative and maturation ponds during periods of intense algal photosynthesis due to rapid removal of dissolved carbon dioxide which favours hydroxyl ions over bicarbonate. The high pH levels found in facultative and maturation ponds result in the removal of substantial amounts of ammonia by volatilization.

Pano and Middlebrooks (1982) proposed a model (equations 2.1 and 2.2) for the removal of ammoniacal nitrogen in individual facultative and maturation ponds. The model incorporates values for hydraulic loading, pH, temperature and coefficients derived from empirical data. The equations assume first order removal kinetics and complete mixing. For temperatures below 20°C the equation is:

$$C_e = \frac{C_i}{1 + \left(\left(\frac{A}{Q} \right) (0.0038 + 0.000134T) \exp((1.041 + 0.0044T)(pH - 6.6)) \right)} \tag{2.1}$$

For temperatures above 20°C:

$$C_e = \frac{C_i}{1 + \left(\left(5.035x10^{-3} \frac{A}{Q} \right) (\exp(1.540)(pH - 6.6)) \right)} \tag{2.2}$$

where:

C_e = ammonical nitrogen concentration in pond effluent (mg N/*l*)

C_i = ammonical nitrogen concentration in pond influent (mg N/*l*)

A = pond surface area (m^2)

Q = wastewater flow rate (m^3/day)

T = temperature (°C)

pH = 7.3exp (0.0005**A**) [where **A =** influent alkalinity (mg $CaCO_3$/*l*)]

It should be pointed out that the complete-mix hydraulic flow regime assumed for these equations is not realistic. Assumption of such conditions has been criticized by various researchers (Thirumurthi, 1974; Marecos do Monte and Mara, 1987; Arceivala, 1983; Polprasert and Bhattarai, 1985; Von Sperling, 1999) for its failure to take into account short-circuiting and stagnant regions that typically occur in WSP.

Pano and Middlebrooks (1982) developed the model for regions of temperate climate and found a good fit of the data obtained from pond systems. However, for the model to be applied in areas where warmer climates prevail requires caution because the model relies on extrapolation of data for its validity. A more reasonable approach might have been to develop more tentative equations that incorporated data from areas with warmer climates, which could be applied globally. That said, Abis (2002), citing Soares (1996), argues that Pano and Middlebrooks's (1982) equation found good agreement with data from ponds in northeast Brazil in predicting the ammoniacal nitrogen of the effluent of the tested four WSP.

2.5.1.2 Ammonia assimilation

Algal uptake of ammoniacal nitrogen is suggested as another major pathway for ammonia removal (Ferrara and Avci, 1982). Ammonia is the preferred source of nitrogen for algae and has been shown to be taken up in preference to nitrate (Konig et al. 1987).

Ammonia taken up by algae is converted to organic nitrogen, which flows out or settles to the bottom of WSP. Anaerobic digestion of sludge may deliver ammonia back to the water column. About 20–60% of the algal biomass is non-biodegradable and the nitrogen associated with this fraction is expected to remain in the sediment (Mara and Pearson, 1986). The extent of ammonia assimilation into algal biomass depends on the biological activity in the system and is affected by temperature, organic load, retention time and wastewater characteristics (US Environmental Protection Agency, 1983).

2.5.2.3 Biological nitrification

Nitrification of ammonia is an aerobic, chemoautotrophic process that converts ammonia to nitrite and then nitrate. Evidence for nitrification is found by a decreasing ammonia concentration accompanied by an increasing nitrate concentration. There is little evidence for nitrification in facultative and maturation ponds because the concentrations of both nitrate and nitrite are usually very low in both inlets and outlets (Reed, 1985). It is argued that if nitrification were taking place, some accumulation of nitrate would be expected in the surface aerobic layers. Mara and Pearson (1998) conclude that nitrification is unlikely to occur in facultative and maturation ponds due to the low density of nitrifying bacteria found in the aerobic zone. This is thought to be due to the absence of physical attachment sites in the aerobic zone and possible inhibition by pond algae.

2.5.2 Phosphorus

Removal of total phosphorus in WSP depends on the amount of phosphorus leaving the pond water column and entering the pond sediments. It is suggested that phosphorus removal is due to sedimentation of phosphorus as organic phosphorus in the algal biomass and precipitation as inorganic phosphorus such as hydroxyapatite at pH levels above 9.5 (Mara, 1997).

It has been thought that the most efficient way to remove phosphorus is to increase the number of maturation ponds in order to facilitate the algal uptake of phosphorus, which subsequently becomes immobilized in the sediment. Houng and Gloyna (1984) developed a first-order plug-flow model for phosphorus removal in WSP. However, the model has been criticized by Mara and Pearson (1986) for not being practical for design purposes. The model shows a direct relationship between BOD removal and phosphorus removal. It shows that 45% of phosphorus can be removed in WSP if the BOD removal is 90%.

2.6 Advantages of WSP

Having appraised the literature on treatment of wastewater in WSP, the advantages of WSP are now presented. Mara (1976) lists the advantages WSP as follows:

- WSP are very efficient in treating wastewater. Pond systems have been shown to achieve a high degree of BOD removal, excreted pathogen removal (bacteria, virus, protozoa and helminths). This enables the WSP effluent to be reused for unrestricted crop irrigation, restricted crop irrigation and fishpond fertilization (WHO, 1989).
- WSP are a flexible treatment process. Ponds can support hydraulic and organic shock loads. Moshe et al. (1972) found that WSP perform satisfactorily when presented with high concentrations of heavy metals (up to 30mg/l).
- WSP are simple to construct, operate and maintain. They do not need electrical energy for their operation. They utilize the sun as the source of energy that is required by both algae and bacteria.

- Building costs for WSP are low (Arthur, 1983). The only capital required is the cost of excavation and ground compaction. The operational costs of WSP are also low as the cost of manual labour is low in developing countries.

The principal drawback of WSP is that they require large areas of land. However, in many countries with warm climates where WSP could be a useful way to treat wastewater, land is often readily available.

In summary, this chapter has presented how wastewater treatment is carried out in WSP for the removal of BOD, excreted pathogens and nutrients. It has shown how a well-designed and operated system of anaerobic, facultative, and maturation ponds can function satisfactorily and as intended. The chapter has shown how anaerobic ponds play a significant role. In conclusion, a WSP system is a low cost technology, appropriate for the warm climate areas of developing countries. However, WSP can only be efficient in treating wastewater if they are well designed and properly operated. Anaerobic ponds should always be included in WSP because they significantly reduce the overall land requirement. Furthermore, anaerobic ponds are effective in removing *Vibrio cholerae*, the classic pathogen that has shown resistant to the otherwise hostile environmental conditions in facultative and maturation ponds. This is important if the effluent is to be reused for unrestricted crop irrigation. An anaerobic pond that has been well designed and which is operated properly should not release odours.

CHAPTER 3

TRADITIONAL DESIGN METHODS FOR WSP

3.1 Introduction

Arthur (1983) reviewed the design methods and performance of WSP in six developing countries. He reported that WSP have been grossly over-designed and the designs are not responsive to the incremental growth encountered in developing countries. The consequence of employing poor process design methods is the unsatisfactory performance of WSP, as reported by Mara et al. (1992). Furthermore, this represents an uneconomic use of the land needed for a WSP system. Procurement of land is often the major capital investment for a WSP scheme.

Mara et al. (1992) highlighted the status and performance of existing WSP in eastern Africa. They observed that these WSP have design and operational problems, resulting from poor process design, poor physical design and unsatisfactory operation and maintenance.

The US Environmental Protection Agency (1983), Reed at al. (1988) and Shilton and Harrison (2003) have suggested that the hydraulic design of WSP needs to be given serious consideration if the pond system is to perform satisfactorily. They observed that poor hydraulic design reduces the theoretical hydraulic retention time due to short-circuiting and the formation of dead spaces. This results in incomplete removal of the wastewater pollutants. The resulting treated effluent then fails to meet the required standards.

It can be concluded that the performance of a WSP system depends on robust process and physical design methods. The process design should assume a realistic hydraulic flow regime that can be achieved by the physical design.

3.2 Design principles of anaerobic ponds

3.2.1 Volumetric organic loading

An empirical approach is the recommended method for designing anaerobic ponds. Anaerobic ponds are normally designed based on permissible volumetric organic loading rate (λ_v) expressed in g/m^3/day of BOD (Arthur, 1983; Mara, 1976; Mara and Pearson, 1986; Meiring et al., 1968). Volumetric organic loading rate and pond temperature have been observed to correlate satisfactorily on a full-scale anaerobic pond experiment. Meiring et al. (1968)

proposed that permissible volumetric organic loading rates for designing anaerobic ponds should be within a range of 100–400 g/m^3 day of BOD to ensure that anaerobic ponds function as intended. Such a range is thought to be safe for temperatures below 10^{o}C and above 20^{o}C. It is suggested that a volumetric organic loading rate of less than 100 g/m^3/day can cause an anoxic reaction in anaerobic ponds. An anoxic pond is a slow rate reactor, which removes BOD relatively slower than an anaerobic pond. The upper limit of 400 g/m^3 day is established to avoid the risk of odour produced by hydrogen sulphide gas (H_2S).

Mara and Pearson (1986) observed that a sulphate (SO_4) concentration of 500 mg/*l* in domestic wastewater is capable of producing odour if a volumetric organic loading rate of 400 g/m^3 day is attained. They suggested that the volumetric organic loading rate should be reduced to 300 g/m^3 day. This is thought to provide an adequate margin of safety against odour.

More recent research by Mara et al. (1997a) observed that a volumetric organic loading rate of 300 g /m^3/day is in fact conservative and does not make the most economic use of the available land. They found that the volumetric organic loading rate could be increased to 350 g/m^3/day as long as the sulphate (SO_4) concentration in domestic wastewater is less than 300 mg/*l*. This idea seemed logical as the WHO (1993) has recommended the maximum sulphate (SO_4) concentration in drinking water to be less than 250 mg/*l*. Table 3.1 lists the suitable design values of volumetric organic loading rates at various temperature ranges, the design temperature being the mean temperature of the coldest month.

Table 3.1 Design values of permissible volumetric BOD loading rate

Temperature (^{o}C)	Volumetric BOD loading (g/m^3 day)
<10	100
10 – 20	$20T$ -100
20 – 25	$10T$ +100
>25	350

T = temperature

Source: Mara and Pearson (1986) and Mara et al. (1997a).

The process design of anaerobic pond volume is related to the BOD of the raw wastewater (L_i, mg/l), the mean wastewater flow (Q, m^3/day) and the volumetric organic loading rate (λ_v) by equation 3.1:

$$V = \frac{L_i Q}{\lambda_V} \tag{3.1}$$

Traditional process design methods have considered the input design parameters of equation 3.1 as deterministic single average values. Arthur (1983), Campos and Von Sperling (1996) and Mara et al. (1992) have observed that such input design parameters increase when the level of income in a community improves.

Traditional design of anaerobic ponds based on deterministic single values of the input design parameters is considered an unrealistic approach. It results in either overdesign or underdesign of anaerobic ponds such that the available land and project funds for procuring WSP are not used economically. Von Sperling (1996) has suggested that input design parameters should be treated as a range due to uncertainty of their exact values. This considered to be a more realistic approach. Furthermore, it has been suggested that designs based on a range of the input design parameters can also incorporate realistic safety factors.

The traditional method for process design establishes a factor of safety for anaerobic ponds based on the temperature in coldest month (Mara, 1976; Arthur, 1983). The resulting volumetric organic loading rate (Table 3.1) is expected to be low. This requires substantial land (equation 3.1) for constructing anaerobic ponds. If the design temperature can be considered as a range with a minimum value during coldest month to a maximum value during hottest month, the volumetric organic loading rate can be manipulated to vary from 100–350 g/m^3/day during the design stage to follow the temperature pattern. Random values of the volumetric organic loading rate can be selected to design the anaerobic pond volume. The final area of the anaerobic pond can then be calculated based on the acceptable risk and available cost. This design approach is efficient and is based on realistic factors of safety.

3.2.2 BOD removal

The design of an anaerobic pond for BOD removal is based on an empirical approach. Mara and Pearson (1986) and Mara et al. (1997a) have developed an empirical relationship between pond temperature and BOD removal in anaerobic ponds. It has been observed that BOD removal in an anaerobic pond is directly proportional to pond temperature. Marais (1970) showed that at a pond temperature of greater than 15°C, methanogenesis is very efficient in removing BOD. Based on such knowledge, Mara and Pearson (1986) and Mara et al. (1997a) proposed the relationship of the design temperature and design BOD removal in anaerobic pond as shown in table 3.2.

Table 3.2 Design of percentage BOD removal in anaerobic ponds at various temperatures

Temperature (°C)	BOD removal (%)
<10	40
10 - 20	$2T+20$
20 - 25	$2T+20$
>25	70

T = temperature

Source: Mara and Pearson (1986) and Mara et al. (1997a).

The traditional method for process design considers the percentage BOD removal and design temperature in the coldest month as deterministic average single values. This approach fails to take account of the efficient BOD removal that occurs in the hottest month of the summer season. In addition, traditional design methods treat the influent BOD as an exact single value. It has been argued by Campos and Von Sperling (1996) that BOD concentrations in raw wastewater vary significantly, with affluent communities producing more BOD than poor communities. This suggests that the influent BOD into facultative ponds should also be treated as a range and not a deterministic average single value.

BOD removal in anaerobic ponds depends on various environmental conditions, including the quality of the raw wastewater. If the pH of the raw wastewater is beyond the permissible range of ~6.6–7.8, BOD removal by methanogenic bacteria will be reduced significantly. This will consequently increase the influent BOD into the facultative pond. This presents a problem for traditional methods of process design, which treat input design parameters as deterministic single values. The best approach to manage the uncertainty of BOD removal in anaerobic ponds, as suggested by Von Sperling (1996), is to treat the design temperature as a range and this will provide a range of values for the percentage BOD removal, which in turn will result in a range of values for the influent BOD entering the facultative pond.

Mara et al. (1997b) developed a model of BOD removal in anaerobic ponds in Kenya, based on first order kinetics. They assumed a completely mixed hydraulic flow regime in the anaerobic pond based on a hydraulic retention time of either less than or greater than 2 days. Their first-order BOD kinetics model using regression analysis is as follows:

$$K_{BOD} = 0.4416T - 7.286 \qquad \text{for } HRT < 2 \text{ days} \tag{3.2}$$

$$K_{BOD} = 0.0207T - 0.202 \qquad \text{for } HRT > 2 \text{ days} \tag{3.3}$$

where

HRT = hydraulic retention time

T = temperature (°C)

They suggested that the effluent BOD could be determined from the following equations:

$$\theta_a = \frac{L_i}{\lambda_v} \tag{3.4}$$

$$L_e = \frac{L_i}{\left(1 + K_{BOD}\theta_a\right)} \tag{3.5}$$

where

θ_a = mean hydraulic retention time in anaerobic pond (days)

L_i = influent BOD (mg/*l*)

L_e = effluent BOD into facultative pond (mg/*l*)

λ_v = volumetric organic loading rate (g/m^3/day)

K_{BOD} = first order BOD removal constant rate (d^{-1})

This proposed model could only be applied in Kenya as the extrapolation of K_{BOD} would be inaccurate in regions with temperatures outside 9.5–25.4°C. Design of the influent BOD into facultative ponds using equations 3.2–3.5 based on deterministic single values of temperature, influent BOD into anaerobic pond and the first-order BOD removal constant is not a realistic approach.

Silva et al. (1996) developed an empirical model of volumetric removal of BOD (g/m^3 day) with volumetric BOD loading (g/m^3 day) in northeast Brazil. Their proposed model is as follows:

$$(VRBOD) = 0.678(VLBOD) \tag{3.6}$$

$r = 0.948$

where

VRBOD = volume removal of BOD g/m^3/day (g BOD g/m^3/day)

VLBOD = volume load in (g BOD g/m^3/day)

r = coefficient of correlation

Silva et al. (1996) suggested that the design percentage of BOD removal for anaerobic ponds in northeast Brazil at 25^{o}C could be 70%. The suggested model could only be realistic for determining the percentage BOD removal in anaerobic ponds as long as the design temperature (25 ^{o}C) is constant.

However, the design models of BOD removal for anaerobic ponds in Kenya and northeast Brazil require the designer to use deterministic single average values of design temperature, volumetric organic loading rate and influent BOD concentration. The drawback of designing anaerobic ponds using traditional process design methods in these countries or in regions with a similar climate as recommended by the authors is that the models fail to manage the uncertainty of such input design parameters and the coefficients used in these models. Designing an anaerobic pond for a given percentage BOD removal based on deterministic single average value is a conservative approach, which results in the suboptimal use of the available land and project funds.

3.3 Design principles of facultative ponds

The design of facultative ponds focuses on BOD removal. Modelling of the biochemical processes within facultative ponds has been attempted in an effort to provide accurate estimates of the surface BOD loading rate, so that the satisfactory performance of the ponds can be ensured. More recently, it has been considered that hydraulic transport processes also have an important influence on BOD removal in facultative ponds.

Mara (1976) and Marecos do Monte and Mara (1987) describe how the design of facultative ponds is currently based on rational and empirical approaches. The empirical design approach is based on correlating performance data of existing WSP. The rational design approach models the pond's performance by using kinetic theories of biochemical reactions in association with the hydraulic flow regime.

These design methods have been noted to have limitations. The empirical method fails to extrapolate pond performance data in regions which have different climate and environmental conditions to regions in which the empirical relationships have been established. The rational design method fails to determine confidently the first order BOD kinetics, which have been observed to vary widely. In addition, to determine dispersion numbers accurately requires tracer studies to be done in existing WSP. It has been suggested that dispersion numbers vary significantly – from zero to infinity – depending on environmental conditions, the mode of pond mixing and the hydrodynamics of the incoming flow (Thirumurthi, 1974).

3.3.1 Design models

Mara (1976) has criticized the Herman and Gloyna (1958) equation and the Gloyna (1976) equation as inappropriate models for designing facultative ponds in warm climates regions. These equations are not reviewed in this chapter.

3.3.1.1 Marais and Shaw (1961) model

Marais and Shaw (1961) proposed a model of designing facultative ponds based on first order kinetics in a complete-mix reactor. In a complete-mix situation, the in-pond concentration is assumed to be the same and equal to the effluent BOD concentration. The basic relationship is as follows:

$$\frac{L_e}{L_i} = \left[\frac{1}{1 + K_{BOD_C}\theta_f}\right]^n \tag{3.7}$$

$$K_{BOD_C} = 0.3(1.05)^{T-20} \tag{3.8}$$

where

L_e = effluent BOD (mg/l)

L_i = influent BOD (mg/l)

K_{BOD_C} = completely mixed flow first order rate constant for BOD removal (d^{-1})

θ_f = mean hydraulic retention time in facultative pond (days)

n = number of ponds in series

T = mean temperature of the coldest month (^{o}C)

Utilization of this equation requires the assignment of L_e. Mara (1976) recommends that L_e should be in the range of 50–100 mg/l (usually 70 mg/l). Equations 3.7 and 3.8 enable the mean hydraulic retention time to be calculated. The pond area is then calculated by using equation 3.9 as follows:

$$A_f = \frac{Q\theta_f}{H} \tag{3.9}$$

where

Q = design flow (m^3/day)

H = pond depth in m (1.5 - 2m)

A_f = area of facultative pond (m^2)

Mara (1976) and Arthur (1981) suggested that the assumptions made by Marais and Shaw (1961) that the first-order rate for BOD removal remains constant over the hydraulic retention time is not realistic. They suggested that the BOD removal constant rate retards exponentially with hydraulic retention time.

The complete-mix hydraulic flow regime has been criticized by various researchers for representing an unrealistic mixing pattern in facultative ponds. A dispersed hydraulic flow regime has been favoured by Thirumurthi (1974) as the realistic mixing pattern that occurs in facultative ponds.

Wehner and Wilhelm (1956) proposed the dispersion number model for chemical reactors based on hydraulic flow pattern, length and longitudinal dispersion. Their model of dispersion numbers that can be used in facultative pond is shown in equation 3.10:

$$d_f = \frac{D}{ul} \tag{3.10}$$

where

D = coefficient of longitudinal dispersion (m^2/s)

u = flow velocity (m/s)

l = length of fluid travel path from influent to effluent (m)

d_f = dispersion numbers in facultative pond

The equation of Marais and Shaw (1961) assumes that the dispersion number (d_f) of Wehner and Wilhelm's equation 3.10 is infinity in a complete-mix flow. Critical analysis of equation 3.10 shows that complete-mix conditions in a facultative pond can be approximated if the shape of the pond is square. This reduces the product of flow velocity and the length of fluid travel (ul) such that the value of the dispersion number (d_f) is high.

Arceivala (1983), Polprasert and Bhattarai (1985) and Mara et al. (1992) have suggested that secondary facultative ponds should be constructed in a rectangular shape with a high length-to-width (4 –10:1). Such a geometric design of facultative ponds is thought to approximate a plug-flow regime. Design of facultative ponds assuming a complete-mix hydraulic flow regime is unrealistic and leads to overdesign.

The complete-mix hydraulic flow model proposed by Marais and Shaw (1961) assumes that wind mixing and temperature are responsible for complete mixing in facultative ponds. Shilton and Harrison (2003) and Tchobanoglous et al. (2003) argue that wind mixing and temperature alone fail to achieve complete mixing in a wastewater treatment plant. They list the following factors that lead to non-ideal hydraulic flow regimes in facultative ponds:

- Temperature differences withiin the pond
- Wind-driven circulation patterns
- Inadequate mixing
- Axial dispersion caused by advection, velocity differences, turbulent eddies, molecular diffusion that occurs in facultative pond.
-

The above points strongly support the idea that a complete-mix hydraulic flow pattern in facultative ponds cannot be achieved.

3.3.1.2 Plug flow

A plug hydraulic flow regime has also been the basis of a rational approach in designing facultative ponds. This is considered the most efficient approach which ensures that the wastewater pollutants attain the theoretical hydraulic retention time. It tries to minimize short-circuiting and formation of dead spaces. In a plug hydraulic flow regime, every element of the flow leaves the pond in the same order in which it entered.

Reed et al. (1988) proposed a plug hydraulic flow regime to design primary facultative ponds. Their model is:

$$\frac{L_e}{L_i} = e^{-K_{BOD_P}\theta_f} \tag{3.11}$$

where

L_e = effluent BOD (mg/l)

L_i = influent BOD (mg/l)

K_{BOD_P} = plug flow first order rate constant for BOD removal (d^{-1})

θ_f = mean hydraulic retention time in facultative pond (days)

K_{BOD_P} is related to any temperature as follows

$$K_{BOD_P} = K_{BOD_{P20}} (1.06)^{T-20}$$

$K_{BOD_{P20}}$ = reaction rate at temperature of 20°C (d^{-1})

T = mean temperature of the coldest month (°C)

Reed at al. (1988) proposed that $K_{BOD_{P20}}$ depends on the BOD surface loading rate as shown in table 3.3. They suggested that if the value of $K_{BOD_{P20}}$ is not known, a value of $0.1 d^{-1}$ could be confidently adopted.

Table 3.3 Variation of the plug-flow reaction rate ($K_{BOD_{P20}}$) with surface organic loading rate

Organic loading rate (kg/ha day)	$K_{BOD_{P20}}$ (day^{-1})
22	0.045
45	0.071
67	0.083
90	0.096
112	0.129

Source: Reed at al. (1988)

The plug-flow model is used to calculate the retention time required for specified BOD removal requirements. If the flow rate is known, the required volume may be calculated by multiplying the flow rate and the theoretical retention time.

The limitation of the application of this proposed model in warm climate regions is the limited surface organic BOD loading rate range proposed. This model could be inappropriate in warm climates, where a higher surface organic loading rate is more likely (Mara et al. 1992).

The plug hydraulic flow regime is considered unrealistic WSP because zero longitudinal mixing is impossible to achieve (Thirumurthi, 1969; Thirumurthi, 1974; Marecos do Monte and Mara, 1987; Tchobanoglous et al. 2003). Wehner and Wilhelm (1956) argue that plug flow conditions could only be achieved if the length of liquid travelling in a reactor is close to infinity. The length of most facultative ponds is limited by practical considerations. Efforts have been made to use baffles to increase the length of liquid travel in facultative and maturation ponds (Pearson et al. 1995; Pearson et al. 1996b; US Environmental protection Agency, 1983; Reed at al. 1988). However, an infinite length of liquid travel cannot be attained in practice and the proposed model by Reed et al. (1988) cannot be realized in practice.

Reed et al. (1988) observed the variation of the plug flow BOD removal constant rate ($K_{BOD_{P20}}$) with surface organic loading rate. The first order BOD removal constant rate ($K_{BOD_{P20}}$) dictates the size of the facultative pond. It has been suggested by Reed et al. (1988) that a deterministic value of $K_{BOD_{P20}}$ should be chosen from table 3.3. However, using a deterministic single average value of the first order BOD removal constant ($K_{BOD_{P20}}$) in designing a facultative pond could lead to an unrealistic and uneconomic design of the pond due to the variation of $K_{BOD_{P20}}$.

3.3.1.3 Dispersed flow regime model

Thirumurthi (1969) recommended that ponds be designed as dispersed flow reactors since they are neither plug flow nor completely mixed. He proposed the use of pond dispersion numbers (d_f) and the first order equation of Wehner and Wilhelm (1956). His equations are as follows:

$$\frac{L_e}{L_i} = \frac{4ae^{\frac{1}{2d_f}}}{(1+a)^2 e^{\frac{a}{2d_f}} - (1-a)^2 e^{\frac{-a}{2d_f}}} \tag{3.12}$$

$$a = \sqrt{(1 + 4K_{BOD_D}\theta_f d_f)} \tag{3.13}$$

$$d_f = \frac{D}{vl} \tag{3.14}$$

$$K_{BOD_D} = K_{BOD_{D20}}(1.09)^{T-20} \tag{3.15}$$

where

L_e = effluent BOD (mg/l)

L_i = influent BOD (mg/l)

K_{BOD_D} = dispersed flow first order reaction rate for BOD removal at any temperature (d^{-1})

$K_{BOD_{D20}}$ = dispersed flow first order reaction rate for BOD removal (d^{-1}) at 20^{o}C

θ_f = mean hydraulic retention time in facultative pond (days)

d_f = dispersion numbers

D = coefficient of longitudinal dispersion (m^{2}h^{-1})

v = mean velocity of travel (mh^{-1})

l = mean path length of a typical particle in the pond (m)

T = minimum pond temperature (^{o}C)

Thirumurthi developed a chart to facilitate the use of the complicated equation 3.12 where (K_{BOD_D} x θ_f) is plotted against the percentage BOD remaining in the effluent for dispersion numbers, d_f, varying from zero for a plug flow to infinity for a complete-mix reactor. Dispersion numbers for ponds have been suggested to range from 0.1 to 4, with most values not exceeding 1.0.

The difficulty which is encountered in designing facultative ponds using equation 3.12 lies in the fact that at the design stage the value of the dispersion number (d_f) and the first order reaction rate for BOD removal (K_{BOD_D}) are not known. Thirumurthi proposed that K_{BOD_D} should be developed based on various environmental conditions that are known to be toxic to pond ecology. This is cumbersome and requires expensive laboratory work and the resulting K_{BOD_D} cannot be determined with high level of accuracy. It has been suggested that dispersion numbers can be obtained from tracer studies in existing ponds. However, for designing a new WSP this might be impossible if there are no similar WSP already existing.

Polprasert and Bhattarai (1985) proposed the following equation for a dispersed hydraulic flow model in facultative pond design:

$$d_f = \frac{0.184\left[\theta_f v(W+2H)\right]^{0.489} W^{1.511}}{(LH)^{1.489}} \tag{3.16}$$

where

d_f = dispersion numbers in facultative pond

v = kinematics viscosity of the pond liquid (m^{2}s^{-1})

L = pond length (m)

W= pond width (m)
H = pond depth (m)
θ_f = mean hydraulic retention time in facultative pond (days)

Polprasert and Bhattarai's equation 3.16 and the dispersed flow model equation 3.12 can be used to design a facultative pond by trial and error. The required effluent BOD value can be assigned until the final hydraulic retention time balances the two sides of equation 3.12. This enables the volume of the facultative pond to be determined by the product of design flow and hydraulic retention time.

Mara and Pearson (1986) cautioned the validity of the Polprasert and Bhattarai (1985) equation. It is suggested that it is developed from pilot-scale studies and as such its validity for the full-scale design of a facultative pond is not known.

Arceivala (1983) proposed the model of coefficient of longitudinal dispersion that could be applied simultaneously with the dispersion numbers model of Wehner and Wilhelm (1956) equation.

The equation of Wehner and Wilhelm (1956) for dispersion numbers is shown again below:

$$d_f = \frac{D}{ul}$$

The equations of Arceivala (1983) are as follows:

For baffled facultative ponds of width larger than 30m,

$$D = 33W \tag{3.17}$$

without baffles,

$$D = 16.7W \tag{3.18}$$

For baffled facultative ponds of width less than 10m,

$$D = 11W^2 \tag{3.19}$$

without baffles

$$D = 2W^2 \tag{3.20}$$

where
W = pond width (m)
D = coefficient of longitudinal dispersion (m^2h^{-1})

Arceivala (1983) suggested that the dispersion numbers that could be derived from equations 3.17 - 3.20 could be solved simultaneously with the dispersed flow model of equation 3.10 to determine the hydraulic retention time. The facultative pond area could be derived from the volume determined by the product of design flow and hydraulic retention time.

The limitation of the equation of Arceivala (1983) is that the model cannot be applied to design facultative ponds whose width ranges between 10m and 30m. This could be a problem if the available land only allows for these dimensions.

3.3.1.4 Surface BOD loading

The surface BOD loading method is the recommended approach for designing facultative ponds. According to the US Environmental Protection Agency (1983) and Reed et al. (1988), for every climate there is an appropriate value of surface BOD loading (kg BOD/ha/day) which can be applied to a pond for a given removal efficiency before failure. Most of the biochemical processes in a facultative pond are surface related: sunlight reception, algal growth and aeration. McGarry and Pescod (1970) found that surface loading values give a closer correlation with performance data than volumetric loading values.

3.3.1.5 McGarry and Pescod (1970)

McGarry and Pescod (1970) correlated data from ponds under 143 different climatic conditions and reported that BOD removal in primary facultative ponds was between 70 - 90%. Their statistical modelling of the data found that pond performance was related to surface BOD loadings, with a high correlation coefficient of 0.995. Their regression analysis is given in equation 3.21 as follows:

$$\lambda_r = 10.75 + 0.725\lambda_s \qquad (3.21)$$

where

λ_r = surface BOD removal (kg/ha/day)

λ_s = surface BOD loading (kg/ha/day)

McGarry and Pescod (1970) observed that the maximum surface BOD loading rate (λ_s) which a primary facultative pond could withstand before failure was related to the ambient air temperature by the equation 3.22 as follows:

$$\lambda_{sf} = 60(1.099)^{T} \tag{3.22}$$

where

λ_{sf} = surface BOD loading in facultative pond (kg/ha/day)

T = temperature (^{o}C)

Mara (1987) adapted the McGarry and Pescod failure model by incorporating a factor of safety to ensure the safe design of facultative ponds. Experience of the surface BOD loading rate in Brazil and Europe enabled Mara to propose a global surface loading rate equation. The tentative global equation proposed by Mara (1987) is as follows:

$$\lambda_{sf} = 350(1.107 - 0.002T)^{T-25} \tag{3.23}$$

where

λ_{sf} = surface BOD loading in facultative pond (kg/ha/day)

T = temperature (^{o}C)

The facultative pond area is calculated by using equation 3.24:

$$A_f = \frac{10 L_i Q}{\lambda_{sf}} \tag{3.24}$$

where

A_f = area of facultative pond (m^2)

L_i = influent BOD concentration (mg/l)

Q = mean flow (m^3/day)

The surface BOD loading rate is the recommended empirical design approach that has been used in traditional process design methods. The calculation for the area of a facultative pond depends on the input design parameters of influent BOD (L_i), design wastewater flow (Q), and the surface BOD loading rate, which is a function of temperature.

The traditional process design method treats these input design parameters as deterministic average single values (Arthur, 1983; Mara and Pearson, 1998; Von Sperling, 1996). Earlier studies of WSP by Mara (1976) and Arthur (1983) recognized the variability of some of these input design parameters such as per capita BOD and per capita water requirement.

Design wastewater flow is related to population size, population growth rate, infiltration of ground water and industrial wastewater flow. It has been observed by Von Sperling (1996) that the design wastewater flow should not be treated as a single average value in developing countries because the rate of growth of the population increases rapidly in these countries. Furthermore, most developing countries lack basic design data due to inadequate resources. The design of facultative ponds using average single values of input design parameters can lead to overdesign or underdesign of the facultative ponds because of the inherent uncertainties in the input parameters.

Surface BOD loading rate has been found to be a function of temperature. The traditional process design method uses the mean temperature in the coldest month as the design temperature. It has been suggested that this approach provides a factor of safety (Mara, 1976; Arthur, 1983; US Environmental Protection Agency, 1983). However, this approach requires critical assessment as the temperature changes continuously from the cold season to the hot season each year. The author is of the opinion that surface loading rate should be manipulated to vary from 100kg/haday to 350kg/haday as proposed by Mara (1987) to follow the pattern depicted by the temperature variation. The traditional process design method for facultative ponds is conservative and can result in the uneconomic use of the available land. It is more realistic for the designer to input a range of parameters which can be set with confidence based upon a given level of uncertainty. This is a cost-effective and safe approach (Von Sperling, 1996).

3.3.2 BOD removal in facultative ponds

Modelling of BOD removal in facultative ponds has been based on empirical design approaches. McGarry and Pescod (1970) developed a model of BOD removal in primary facultative pond using regression analysis. In their model, they observed that surface BOD removal correlated well with surface BOD loading. Their model is presented in equation 3.21 as follows:

$$\lambda_r = 10.75 + 0.725\lambda_{sf}$$

where

λ_r = surface BOD removal (kg/ha/day)

λ_{sf} = surface BOD loading (kg/ha/day)

The graphical presentation of the regression line given by McGarry and Pescod (1970) with 95% confidence interval shows that the constant term “10.75” is close to zero. Research by Mara et al. (1997) confirmed the validity of the BOD removal pattern suggested by McGarry and Pescod (1970). It is suggested that the combined BOD removal of anaerobic and secondary facultative ponds in series is greater than 80% for temperatures above 20^{o}C. It is added that for temperatures below 20^{o}C, the combined BOD removal by anaerobic and secondary facultative pond could be assumed to be ~70%.

3.4 Design principles of maturation ponds

Marais (1974) proposed a design model for a series of ponds for faecal coliform removal. He assumed that faecal coliform removal could be modelled by first-order kinetics in a completely mixed reactor. His proposed model is presented in equation 3.25 as follows:

$$\frac{N_e}{N_i} = \frac{1}{\left(1 + K_{FC_T}\theta\right)} \tag{3.25}$$

where

N_e = number of effluent faecal coliform per 100ml

N_i = number of influent faecal coliform per 100ml

K_{FC_T} = first order rate constant for faecal coliform removal (d^{-1})

θ = retention time (days)

For a series of anaerobic, secondary facultative, and maturation ponds, Marais (1974) model is presented in equation 3.26 as follows:

$$\frac{N_e}{N_i} = \frac{1}{\left(1 + K_{FC_T}\theta_a\right)\left(1 + K_{FC_T}\theta_f\right)\left(1 + K_{FC_T}\theta_m\right)^n} \tag{3.26}$$

where

the subscripts a, f, m refer to the anaerobic, facultative and maturation ponds, and

n = number of maturation ponds.

Marais suggested that the most efficient pond configuration would be achieved if all the maturation ponds were of equal size, such that they had the same hydraulic retention time. However, due to topographical limitations, the size of maturation ponds cannot always be the same, in which case the Marais model is modified into equation 3.27 as follows:

$$\frac{N_e}{N_i} = \frac{1}{\left(1 + K_{FC_T}\theta_a\right)\left(1 + K_{FC_T}\theta_f\right)\left(1 + K_{FC_T}\theta_{m1}\right)\left(1 + K_{FC_T}\theta_{m2}\right)\left(1 + K_{FC_T}\theta_{mn}\right)} \tag{3.27}$$

Marais (1974) suggested that the faecal coliform die-off rate (K_{FC_T}) is temperature-dependent. His model for the faecal coliform die-off rate is presented in equation 3.28 as follows:

$$K_{FC_T} = 2.6(1.19)^{T-20} \tag{3.28}$$

where T = temperature (^{o}C)

Three conditions are set to ensure that maturation ponds are designed satisfactorily. These conditions are as follows:

1. $\theta_m < \theta_f$
2. $\theta_m > \theta_m^{\min}$

where $\theta_m^{\min}$ = 3–5 days

3. $\lambda_{sm1}(BOD) \leq 0.75\lambda_{sf}(BOD)$

where

θ_m = hydraulic retention time in each maturation ponds (days)

θ_f = hydraulic retention time in secondary facultative pond (days)

$\theta_m^{\min}$ = minimum hydraulic retention time in maturation ponds (days)

λ_{sm1} = surface BOD loading in first maturation pond (kg/ha/day)

λ_{sf} = surface BOD loading in facultative pond (kg/ha/day)

It is proposed by Mara (1997) that an assumption can be made for an 80% BOD (L_i, mg/l) reduction in anaerobic and facultative ponds at temperatures greater than 20^{o}C and a 70% BOD (L_i, mg/l) reduction for temperatures less than 20^{o}C. Mara (1997) proposed the determination of the minimum hydraulic retention times in the first maturation pond and second and subsequent maturation ponds by using equation 3.29 and 3.30, respectively, as follows:

$$\theta_{m1}^{\min} = \frac{10L_i H}{0.75\lambda_{sf}} \tag{3.29}$$

$$\theta_m = \frac{\left[\dfrac{N_i}{N_e\left(1+K_{FC_T}\theta_a\right)\left(1+K_{FC_T}\theta_f\right)\left(1+K_{FC_T}\theta_{m1}^{\min}\right)}\right]^{\frac{1}{n}} - 1}{K_{FC_T}} \tag{3.30}$$

where

θ_a = hydraulic retention time in anaerobic pond (days)

θ_m = hydraulic retention time in each maturation pond (days)

θ_f = hydraulic retention time in secondary facultative pond (days)

$\theta_{m1}^{\min}$ = minimum hydraulic retention time in first maturation pond (days)

H = design depth of maturation pond (m)

L_i = influent BOD concentration in first maturation pond (mg/l)

Equation 3.30 is solved for n = 1, 2, 3 until $\theta_m < \theta_m^{\min}$ (3–5 days). A choice is made of θ_m and n for which their product is a minimum as this is considered to provide the least land requirement.

The outlined traditional process design methods for maturation ponds have been strongly criticized by various researchers as being unrealistic and unsafe. Thirumurthi (1974), Arceivala (1983), Polprasert and Bhattaria (1985), Marecos do Monte and Mara (1987), and Von Sperling (1999) have suggested that a completely mixed hydraulic flow regime cannot be realized in maturation ponds. They suggest that a dispersed hydraulic flow regime is the realistic non-ideal flow that simulates the real hydraulic flow pattern in WSP.

The complete-mix hydraulic flow regime proposed by Marais (1974) can only be achieved in maturation ponds if the length-to-width ratio is close to unity (Arceivala, 1983; Polprasert and Bhattaria, 1985; Von Sperling, 1999). Maturation ponds are geometrically designed to have a high length-to-width ratio (up to 10:1) such that they can simulate a plug-flow regime to enhance their performance (Mara et al. 1992). Under these conditions, the complete-mix hydraulic flow regime suggested by Marais (1974) in modelling faecal coliform removal is not realistic. Shilton and Harrison (2003) have suggested that an ideal hydraulic flow regime in wastewater reactors cannot be achieved due to short-circuiting and dead spaces that are produced by changes in environmental conditions such as temperature and wind-driven patterns. They add that changes in the influent flow of wastewater produce turbulence and eddies which are responsible for dispersing faecal coliforms.

The temperature-dependent model proposed by Marais (1974) for faecal coliform removal has been considered unrealistic by various researchers. Parhad and Rao (1974), Polprasert et al. (1983), Qin et al. (1991), Saqqar and Pescod (1992), Mayo, (1995), Sarikaya and Saatci (1987a), Curtis and Mara (1994) and Von Sperling (1999) have suggested that complex environmental conditions are responsible for faecal coliform removal in WSP. They propose that surface COD loading rate, concentration of pond algae, dissolved oxygen concentration, pH, light intensity and pond depth all play a significant role in killing excreted pathogens in WSP.

Faecal coliform removal models proposed by Polprasert et al. (1983), Qin et al. (1991), Saqqar and Pescod (1992), Mayo (1995) and Sarikaya and Saatci (1987a) have been criticized by Curtis et al. (1992) because the models do not include the combination of dissolved oxygen and sunlight intensity, which have been found to be responsible for faecal coliform removal in WSP. Studies by Cutis and Mara (1994) have demonstrated the removal of faecal coliforms in WSP by the effects of photo-oxidation and high pH (> 9.4).

Traditional process design methods for maturation ponds use input design parameters of influent faecal coliform, hydraulic retention time, temperature, and faecal coliform die-off rate as average single values. Mara (1997), Mara et al. (1992) and Arthur (1983) have suggested that influent faecal coliform levels should be defined as a range. They suggested that a typical range of faecal coliforms in raw wastewater is 10^7–10^8 per 100 ml.

Faecal coliform removal models proposed by Polprasert et al. (1983), Qin et al. (1991), Saqqar and Pescod (1992), Mayo (1995), Sarikaya and Saatci (1987a), Curtis and Mara (1994) and Von Sperling, (1999) depend on various environmental conditions which change continuously in WSP. A dispersed hydraulic flow regime has been found to influence the hydraulic retention time of faecal coliforms. Short-circuiting and dead spaces reduce the design hydraulic retention time. These suggested factors influence the faecal coliform die-off rate.

Traditional process design methods for maturation ponds fail to manage the uncertainty of the input design parameters, which have been demonstrated to vary in reality. The design of maturation ponds as suggested by Marais (1974) based on deterministic average value of influent faecal coliform and temperature could consequently lead to a potentially unsafe system in which the practice of unrestricted crop irrigation with the final treated effluent could pose a public health risk. It has been suggested by Mara et al. (1997b), Pearson et al.

(1995), Saqqar and Pescod, (1992) and Von Sperling, (1999) that Marais's model overestimates the actual faecal coliform die-off rate.

3.5 Design parameters

Mara et al. (1992), Mara (1997) and Arthur (1983) list the most important input design parameters of WSP as temperature, net evaporation, design flow, per capita BOD and faecal coliform concentration. It is suggested that helminth eggs are required if the effluent is to be reused for restricted crop irrigation. These design parameters are subjected to uncertainty.

3.5.1 Temperature

Mara and Pearson (1998) suggest that design temperature should be the mean air temperature in the coldest month. This is considered to provide a small margin of safety as the pond temperature is 2 - 3°C warmer than the air temperature in the cool season. Mara and Pearson (1998) suggest that during the hot season, pond temperatures are higher than the air temperature. This chapter has demonstrated that volumetric BOD loading rate, surface BOD loading and BOD removal in anaerobic and facultative ponds are temperature-dependent. These design parameters attain their maximum values during the hot season and their minimum values during the cold season.

Treating the design temperature as a range with a minimum value of air temperature in the coldest month up to a maximum value of mean air temperature in the hottest month could be an efficient method of designing WSP. Under these conditions, the design values of volumetric BOD loading, surface BOD loading and BOD removal could be arranged to vary between minimum and maximum values, as proposed by Mara (1997). Monte Carlo simulations can be applied at the design stage to allow the design variables to be inputted as a range.

3.5.2 Net evaporation

Net evaporation is defined as evaporation of pond liquid less rainfall. It has been suggested that values for net evaporation should be taken into account in the design of facultative and maturation ponds but not in anaerobic ponds. The net evaporation rates in the months used for selecting the design temperature are those used in designing facultative and maturation ponds.

It is suggested that a hydraulic balance should be carried out for the hottest month (Mara and Pearson, 1998).

Net evaporation is an important design parameter. It affects the mean design flow used for designing facultative and maturation ponds. Furthermore, it affects the hydraulic retention time for which wastewater pollutants will remain in WSP. Evaporation rates are normally high in the hot month of the season and low in the cold month of the season. If the design of WSP is based on a range of net evaporation rates to account for this variation, more economical land use is likely to result. The traditional method for process design uses a single average value of net evaporation rates in the coldest month. During the cold season, the net evaporation rates are low and the net value of the mean flow is relatively higher than is likely to have been the case in a hot month. The consequence is that this approach results in a conservative design of WSP.

3.5.3 Design flow

Mara and Pearson (1998) suggest that where a wastewater treatment plant is in operation, the mean daily flow should be measured. If the plant does not yet exist, a value for the mean daily flow should be estimated, because the size of the WSP and the cost of the system's construction are directly proportional to the flow.

In a case of new WSP where no existing treatment plant exists, the mean daily flow is determined by using equation 3.31 and 3.32 as follows:

$$Q = 10^{-3} kqP_d + I + E \tag{3.31}$$

$$P_d = P_i(1+r)^n \tag{3.32}$$

where
Q = design mean daily flow (m^3/day)
k = wastewater return factor (0.8–0.9)
q = per capita water consumption (l/person/day)
P_d = design population
P_i = initial population
n = design period
r = annual population growth rate expressed as a fraction
I = infiltration of groundwater into a sewer line (m^3/day)

E = industrial wastewater flow (m^3/day)

The design flow model has been shown to consist of various components that are subjected to uncertainties. Population growth in developing countries increases rapidly and it cannot be assumed to remain constant during the design period (Von Sperling, 1996). Estimates of industrial wastewater and infiltration of groundwater cannot be determined with certainty. Per capita water consumption has been known to be variable depending on the social status of the community (Campos and Von Sperling, 1996). Population determination in developing countries cannot be predicted accurately. Developing countries lack reliable data due to limited resources. These uncertainties question the basis of traditional process design methods, which treat design flow as deterministic average single value. The author suggests that design flow in WSP should be treated as a range. Monte Carlo simulations could then be applied to the design of WSP taking random design flow data points within the proposed range. This suggested approach is as recommended by Von Sperling (1996) and represents an effective way of managing uncertainty in the mean flow rates.

3.5.4 Per capita BOD

Mara and Pearson (1998) suggested that BOD might be determined by using 24-hour flow-weighted composite samples in those cases where wastewater exists. If wastewater does not yet exist, BOD should be estimated using equation 3.33 presented as follows:

$$L_i = \frac{1000B}{q} \tag{3.33}$$

where

L_i = wastewater BOD (mg/l)

B = BOD contribution (g/person/day)

q = wastewater flow (l/person/day)

Mara et al. (1992), Mara (1997) and Campos and Von Sperling (1996) argue that BOD increases over time as people get wealthier, with affluent communities producing more BOD than poor communities. The researchers suggested that per capita BOD contribution (B) varies from 30g to 70g in developing countries. The traditional process design method treats per capita BOD as a deterministic single value and this is considered unrealistic for the design of WSP.

3.5.5 Faecal coliforms

Faecal coliform numbers are very important design parameters if the pond effluent is to be reused for unrestricted crop irrigation. Mara and Pearson (1998) suggest that grab samples of the wastewater should be used to measure the faecal coliform concentration if the wastewater exists. They propose that the typical range of faecal coliform in wastewater is 10^7–10^8 per 100ml. Data provided by Mara and Pearson (1998) support the view that faecal coliform concentrations should not be regarded as an average single value when designing WSP, as is the case in the traditional process design method. This can underestimate the worst case of faecal coliform numbers which can pose a public health risk if the treated effluent is to be used for unrestricted crop irrigation.

In summary, this chapter has shown how traditional process design methods are used for WSP whose treated effluent is to be used for unrestricted crop irrigation. The chapter has shown that volumetric organic loading rate and surface BOD loading rate are the principal parameters for empirical methods for designing anaerobic and facultative ponds respectively. The chapter has presented how the Marais (1974) equation is used in traditional design methods for maturation ponds. In conclusion, traditional process design methods are conservative and unrealistic. They result in an uneconomical use of the available land and project funds. They produce treated effluent that could pose public health risk if unrestricted crop irrigation is practised. The central weakness of traditional process design methods is the reliance on input design parameters that are deterministic single average values. Studies have shown that input design parameters are subjected to uncertainty. These uncertainties can be managed if the design parameters are considered as a range. The completely mixed hydraulic reactor assumed by Marais (1974) cannot be achieveded in practice. A dispersed hydraulic reactor mimics the non-ideal flow that occurs in WSP. The faecal coliform die-off rate model proposed by Marais (1974) as temperature-dependent is not a realistic model for predicting faecal coliform removal in WSP. Faecal coliform removal in WSP is due to a combination of environmental conditions that exist in WSP.

CHAPTER 4

MODERN DESIGN METHODS OF WSP

4.1 Introduction

According to the World Health Organization (1989), the practice of unrestricted crop irrigation in warm climates areas can be satisfied by a series of WSP. Such a series of WSP is supposed to have at least maturation ponds that are specifically designed for the removal of excreted pathogens. The equation of Marais (1974) is currently used to design maturation ponds that reach the criteria for unrestricted irrigation.

Various researchers (Thirumurthi, 1974; Arceivala, 1983; Polprasert and Bhattaria, 1985; Von Sperling, 1999) have suggested that this equation does not provide a realistic basis for the design of maturation ponds because it assumes an ideal completely mixed hydraulic flow regime that is not realized in practice. The faecal coliform die-off rate model proposed by Marais (1974) has also faced strong criticism from various researchers (Mayo, 1989; Mayo, 1995; Sarikaya and Saatci, 1987a; Saqqar and Pescod, 1992; Curtis et al. 1992; Curtis and Mara, 1994; Von Sperling, 1999) because of its reliance on simple temperature-dependence. The same researchers have also criticized the assumptions made by Marais that the faecal coliform die-off rate should be the same in anaerobic, facultative and maturation ponds. They suggest that each type of WSP should have different faecal coliform die-off rate, given that the environmental conditions responsible for killing faecal coliforms will be different in each type of pond.

Modern design methods for WSP address these shortcomings. Modern techniques assume a dispersed hydraulic flow regime for modelling faecal coliform removal in facultative and maturation ponds. Instead of the simple faecal coliform die-off rate model proposed by Marais (1974), the modern design methods apply Von Sperling's empirical equations (1999) to model faecal coliform removal in facultative and maturation ponds. This model has been observed to predict satisfactory (R^2 = 95.7%) effluent faecal coliform concentrations in WSP. On the concern of the application of the same faecal coliform die-off rate in each type of WSP, modern design methods use Mara's empirical equation (2002) to predict the faecal coliform concentration in the effluent of anaerobic ponds.

Traditional design methods are considered an unrealistic, conservative and unsafe approach for WSP (Chapter 3). The principal problem is that the input parameters are treated as deterministic average single values, whereas in reality these parameters have been shown to have uncertainties (Chapter 3). The coefficients and the input design variables used in the empirical equations of Von Sperling (1999) and Mara (2002) are also subjected to uncertainties. Modern approaches to WSP design address this problem by treating the input design parameters as a range. This is done by using a Monte Carlo simulation, which selects randomly design values within the defined range.

4.2 The hydraulic dispersed flow regime

Marais (1974) developed the completely mixed hydraulic flow regime in order to simulate the hydraulic transport of excreted pathogens in maturation ponds. He assumed that faecal coliform removal in maturation ponds follows first order kinetics and are completely mixed. Marais's faecal coliform removal model for a series of anaerobic, facultative and maturation ponds is presented in equation 3.27 and 3.28 (Chapter 3) as follows:

$$\frac{N_e}{N_i} = \frac{1}{\left(1 + K_{FC_T}\theta_a\right)\left(1 + K_{FC_T}\theta_f\right)\left(1 + K_{FC_T}\theta_m\right)^n}$$

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

where

the subscripts a, f, m refer to anaerobic, facultative and maturation ponds

N_e = number of effluent faecal coliform concentration per 100 ml

N_i = number of influent faecal coliform concentration per 100 ml

K_{FC_T} = first-order constant rate for faecal coliform removal (d^{-1})

θ = mean hydraulic retention time (days)

n = number of maturation ponds

T = temperature (^oC)

Various researchers (Thirumurthi, 1974; Arceivala, 1983; Polprasert and Bhattaria, 1985; Von Sperling, 1999) have criticized Marais's equation 3.27 as being unrealistic from a hydraulic point of view. They suggest that a completely mixed hydraulic flow regime could not be achieved in WSP. They propose that a dispersed hydraulic flow regime should be used when modelling the hydraulic performance of WSP.

Thirumurthi (1969), Arceivala (1983) and Polprasert and Bhattaria (1985) suggested that the geometrical aspects of WSP and the inlet and outlet configuration are the most important physical factors that can be manipulated to approximate the ideal hydraulic flow regime in WSP. Wehner and Wilhelm (1956) developed the dispersion number model in a chemical reactor based on hydraulic flow characteristic, length of fluid travel and longitudinal dispersion. Their proposed model of the dispersion number is presented in equation 3.10 (Chapter 3) as follows:

$$d = \frac{D}{ul}$$

where

D = coefficient of longitudinal dispersion (m^2/s)

u = flow velocity (m/s)

l = length of fluid travel path from influent to effluent (m)

d = dispersion numbers

Wehner and Wilhelm (1956) suggested that the completely mixed hydraulic flow regime produces infinite dispersion numbers (d) in a reactor. From a mathematical point of view, under the ideal completely mixed conditions, the denominator (ul) of Wehner and Wilhelm's dispersion number model should be close to zero. The completely mixed condition can be approximated in practice by using a WSP with a square or circular shape (Arceivala, 1983; Polprasert and Bhattaria, 1985).

However, WSP are physically designed to have a rectangular shape with a higher ratio of length to width (up to 1:10) to approximate plug flow (Mara et al. 1992; Shilton and Harrison, 2003). This therefore suggests that the completely mixed hydraulic flow regime assumed by Marais (1974) cannot be realized in practice since WSP are geometrically constructed to produce plug flow.

The dispersion model of Wehner and Wilhelm (1956) suggests that a plug flow regime occurs in WSP if the longitudinal dispersion number in a reactor is zero. This is attained if the length of the fluid travel in equation 3.10 (the denominator, ul) is infinite. Clearly, this cannot be achieved in practice. Although transverse, horizontal, evenly spaced baffles have been suggested as a way to increase the length of the fluid travel (Shilton and Harrison, 2003), infinity cannot be achieved.

The author is of the opinion that the ideal hydraulic flow regime cannot be realized in WSP. The author agrees with the views of the various researchers (Thirumurthi, 1969; Arceivala, 1983; Marecos do Monte and Mara, 1987; Polprasret and Bhattaria, 1985; Agunwamba et al. 1992; Von Sperling, 1999) that a dispersed hydraulic flow regime should be used when modelling the hydraulic performance of WSP. Short-circuiting and dead space formation have been suggested to occur even in a well-designed and constructed WSP (Shilton and Harrison, 2003). These phenomena are thought to account mainly for the non-ideal flow pattern in WSP.

Recently, Shilton and Harrison (2003) and Tchobanoglous et al. (2003) have shown that wind mixing and temperature effects cause the non-ideal hydraulic flow pattern in WSP. It is suggested that temperature differences result in stratification due to density differences within the pond water. This often occurs when the inflowing wastewater stream is colder or warmer than the wastewater in the pond. One consequence of stratification is that the incoming wastewater flows directly to the pond outlet in a fraction of the theoretical hydraulic retention time without mixing into the full volume of the WSP. This is thought to be the cause of the short-circuiting in WSP.

Wind-driven circulation patterns have also been suggested to cause a dispersed hydraulic flow (Shilton and Harrison, 2003; Tchobanoglous et al., 2003). It is suggested that wind shear causes wastewater in WSP to circulate vertically along the pond depth. Shilton and Harrison (2003) argue that the vertical circulation of pond water increases when the horizontal momentum of the inlet flow is low. The vertical circulation of the pond water transports a portion of the incoming wastewater flow to outlet in a fraction of the actual hydraulic retention time.

Shilton and Harrison (2003) have demonstrated that diagonal positioning of the inlet and outlet configuration in WSP cannot eliminate short-circuiting. They suggested that the horizontal momentum of the inlet flow causes the circulation of the influent wastewater horizontally, which reaches the pond outlet within a fraction of theoretical hydraulic retention time. This is suggested as another cause of the non-ideal hydraulic flow pattern in WSP. The researchers suggest that evenly spaced, horizontal transverse baffles across 70% of the pond width are an effective means of reducing short-circuiting. Such an arrangement of baffles is thought to increase the length of the fluid travel and reduce the longitudinal dispersion such that plug flow pattern is approximated.

Tchobanoglous et al. (2003) have further suggested that the daily fluctuation of the incoming wastewater flow into wastewater treatment plants cause turbulent eddies to develop. It is proposed that these turbulent eddies and molecular diffusion cause the dispersion of wastewater pollutants within WSP.

Thirumurthi (1969) suggested that the design of WSP should based on a dispersed hydraulic flow regime as it simulates the real hydraulic flow pattern in WSP. In order to facilitate the design of WSP based on dispersed hydraulic flow regime, Thirumurthi developed a chart based on the dispersion number model of Wehner and Wilhelm (1956) in which the ideal and non-ideal flow mixing pattern are presented based on their dispersion numbers. He demonstrated that the dispersed hydraulic flow regime prevails in WSP with dispersion number (*d*) values ranging from 0.0625 to 4. Thirumurthi measured the dispersion numbers in full-scale WSP by carrying out tracer studies. He found dispersion numbers in WSP of 0.121 and this validated his dispersion chart. These data appear to confirm that the mixing pattern in WSP is neither plug flow nor completely mixed flow.

Finney and Middlebrooks (1980) confirmed that the dispersed hydraulic flow regime occurs in WSP. They used tracer studies to measur the dispersion numbers in full-scale WSP in Corinna, a town in the state of Utah in the US. They found the dispersion numbers in full-scale WSP ranged from 0.395 to 1.710. According to the chart of Thirumurthi (1969), the measured dispersion numbers are within the range that indicates a dispersed hydraulic flow regime (0.0625–4).

Polprasert and Bhattarai (1985) developed a dispersion number model to predict dispersion numbers in full-scale WSP. This model was based on data obtained from tracer studies in pilot-scale WSP. The proposed dispersion number model is presented in equation 3.16 (Chapter 3) as follows:

$$d = \frac{0.184\left[\theta v\left(W + 2H\right)\right]^{0.489} W^{1.511}}{\left(LH\right)^{1.489}}$$

where

d = dispersion numbers

v = kinematic viscosity of the pond liquid (m^2s^{-1})

L = pond length (m)

W= pond width (m)

H = pond depth (m)

θ = mean hydraulic retention time (days)

The observed dispersion numbers ranged between 0.115 and 0.215. Their model predicted the dispersion numbers of the pilot scale WSP to range between 0.068 and 1.182. The predicted dispersion numbers and the observed dispersion numbers are within the range (0.0625–4) of the dispersed hydraulic flow regime asuggested by Thirumurthi (1969). The work of Polprasert and Bhattarai (1985) would seem to add further confirmation that the actual flow pattern in WSP can be represented by a dispersed hydraulic flow regime.

Marecos do Monte and Mara (1987) measured dispersion numbers in full-scale WSP in Portugal using tracer studies. They found number varying between 0.371 and 0.595 during summer and winter seasons. According to the chart of Thirumurthi (1969), the measured dispersion numbers are within the range (0.0625–4) representative of a dispersed hydraulic flow regime. This adds more evidence that the dispersed hydraulic flow regime simulates the non-ideal flow pattern in WSP.

Agunwamba et al. (1992) developed a dispersion number model in Nigeria for predicting the dispersion numbers in WSP. The model was developed based on Polprasert and Bhattarai's (1985) pilot-scale WSP data. Their proposed model is presented in equation 4.1 as follows:

$$d = 0.10201\left(\frac{u_*}{u}\right)^{-0.8963}\left(\frac{H}{L}\right)x\left(\frac{H}{W}\right)^{-\left(0.98074+1.38485\frac{H}{W}\right)} \quad (4.1)$$

where
d = dispersion numbers
u_* = shear velocity (m/day)
u = flow velocity (m/day)
W = pond width (m)
H = pond depth (m)
L = pond length (m)

Agunwamba et al. (1992) measured the dispersion numbers of the full-scale WSP (Nsukka ponds) in Nigeria by tracer study. The measured dispersion numbers ranged from 0.110 to 0.213. In order to validate their proposed model, they predicted the dispersion numbers of the full-scale WSP (Nsukka ponds) in Nigeria and the full-scale WSP in Portugal that were studied by Marecos do Monte and Mara (1987). For WSP in Nigeria, their model predicted the dispersion numbers to range from 0.153 to 0.189. For the full-scale WSP studied by Marecos do Monte and Mara (1987) in Portugal, their model predicted the dispersion numbers to range from 0.116 to 0.409. According to the chart of Thirumurthi (1969), both the measured and the predicted dispersion numbers are within the range (0.0625–4) representative of a dispersed hydraulic flow regime. This is further confirmation that a dispersed hydraulic flow regime should be used to model the hydraulic transport of wastewater pollutants in WSP.

The studies outlined above support modern design methods, which assume a dispersed hydraulic flow regime when modelling faecal coliform removal in facultative and maturation ponds.

4.3 Von Sperling's dispersion number model

Von Sperling (1999) developed the dispersion number model for predicting dispersion numbers in facultative and maturation ponds. His proposed empirical equation of dispersion number based on geometrical aspects of WSP is presented in equation 4.2 as follows:

$$d = \frac{1}{\left(\frac{L}{W}\right)} \tag{4.2}$$

where
d = dispersion numbers
L = pond length (m)
W = pond breadth (m)

Von Sperling (2002) compared the accuracy of the four dispersion number models developed by Polprasert and Bhattarai (1985), Agunwamba et al. (1992), Yanez (1993) and Von Sperling (1999). He used Monte Carlo simulations to predict the variation in the dispersion numbers predicted by the four models. It was found that the dispersion number models proposed by Polprasert and Bhattarai (1985) and Agunwamba et al. (1992) were not accurate in predicting the dispersion numbers due to the wide range of the resulting dispersion numbers. The dispersion number models developed by Yanez (1993) and Von Sperling (1999) predicted accurately the dispersion numbers since they produced a narrow range of the resulting dispersion numbers.

The dispersion number models proposed by Polprasert and Bhattarai (1985) and Agunwamba et al. (1992) not only show a weakness by predicting a wide range of dispersion numbers, they also require the designer to assume some design variables such as kinematic viscosity, shear velocity and flow velocity when utilizing the empirical equations. These variables cannot be determined accurately. Furthermore, these design variables depend on other factors such as temperature and the incoming inlet momentum flow, which make accurate predictions even more difficult.

The dispersion number model of Von Sperling (1999) is much simpler to use in design than that of Yanez (1993). Consequently, modern design methods use Von Sperling's dispersion number model to predict the dispersion of faecal coliforms in facultative and maturation ponds.

4.4 Faecal coliform removal models

4.4.1 Environmental conditions in WSP

Marais (1974) developed a model of faecal coliform die-off rate based on temperature only for designing maturation ponds. The faecal coliform die-off rate model is presented in equation 3.28 (Chapter 3) as follows:

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

where
T = temperature (°C)

Marais suggested that the proposed model is valid at a temperature range of 5 - 21°C because the rate at which faecal coliforms die-off increases with temperature within this range. Marais found that at temperatures above 21°C, the faecal coliform die-off rate decreases due to anaerobic conditions arising during the summer season. He proposed that the same model for faecal coliform die-off rate (K_{FC_T}) could be used in all three types of WSP pond: anaerobic, facultative and maturation.

This simple temperature-dependent model has been strongly criticized by many researchers as being an unrealistic and unreliable way to predict the removal of faecal coliforms in WSP (Sarikaya and Saatci, 1987a; Sarikaya et al. 1987b; Mayo, 1989; Saqqar and Pescod, 1992; Curtis et al. 1992; Curtis and Mara, 1994; Mayo, 1995; Von Sperling, 1999).

Parhad and Rao (1974) observed the effective removal of *E.coli* in cultured algae when the pH exceeded 9.4. Pearson et al. (1987b) observed similar faecal coliform removal in a full-scale WSP when the pond pH exceeded 9.0. These observations show that pH is a significant variable that should be incorporated into any proposed model for faecal coliform removal.

Polprasert et al. (1983) observed that algae concentration, influent COD loading and ultra violet rays affect the removal of faecal coliforms in WSP. They argued that the model proposed by Marais (1974) overestimates the prediction of faecal coliform removal in WSP because of its reliance solely on temperature.

Pearson et al. (1995), Pearson et al. (1996b), Mayo (1989), Sarikaya et al. (1987b), Saqqar and Pescod (1991) and Von Sperling (1999) observed that pond depth also affects faecal coliform removal in WSP. They suggested that shallow ponds are more effective in removing excreted pathogens than deeper ponds. This finding suggests that the depth of the pond should also be a factor in any model for predicting the removal rates of faecal coliforms in WSP.

Curtis et al. (1992) and Curtis and Mara (1994) have explained how environmental factors in facultative and maturation ponds contribute to the removal of faecal coliforms. They observed that sunlight in combination with dissolved oxygen and pH provides an effective mechanism for removing faecal coliforms from facultative and maturation ponds through a process of photo-oxidation. They observed that sunlight alone could not remove faecal coliform if not complemented by a high concentration of dissolved oxygen. These authors argue, therefore, that a combination of sunlight and oxygen is more important in removing faecal coliform than ultra violet rays, as suggested by many researchers.

4.4.2 The limitations of some of the proposed faecal coliform removal models

Sarikaya and Saatci (1987a), Qin et al. (1991), Saqqar and Pescod (1992) and Mayo (1995) have all developed models for predicting faecal coliform removal in WSP based on environmental factors. Each models use some of the following variables: temperature, pH, pond depth, sunlight intensity, soluble BOD_5 and algae concentration. The common variable used in every model is sunlight intensity. None of the models includes dissolved oxygen concentration. Curtis and Mara (1994) suggest that for this reason the models are deficient when attempting to arrive at effective designs of maturation ponds, given that sunlight must work in tandem with dissolved oxygen to remove faecal coliforms.

4.4.3 The Polprasert, Dissanayake and Thanh faecal coliform removal model

Polprasert et al. (1983) developed a faecal coliform removal model for maturation ponds based on a dispersed hydraulic flow regime. The model proposes that faecal coliform removal depends on temperature, organic areal loading and the concentration of algae in the pond according to equation 4.3 as follows:

$$e^{K_{FC_T}} = 0.716(1.0281)^T (1.0016)^C (0.9994)^{\lambda_a} \qquad (4.3)$$

where

K_{FC_T} = faecal coliform die-off rate (d^{-1})

C = algae concentration (mg/l)

T = temperature (^oC)

λ_a = areal COD loading (kg/ha/day)

While Polprasert et al. (1983) did measure the sunlight intensity, pH and dissolved oxygen concentration, they did not include these variables in their model (equation 4.3) because they reasoned that these factors are adequately reflected by the concentration of algae in the pond – a variable that is included in the model.

However, the model of Polprasert et al. (1983) cannot be relied upon to produce a safe design of maturation ponds whose treated effluent is intended for unrestricted crop irrigation. The model requires some of the variables – such as algal concentration – to be assumed. This is something that is difficult to do with a high degree of accuracy. Algal concentration varies widely throughout the seasons as the climatic conditions fluctuate.

4.4.4 The Curtis and Mara faecal coliform removal model

Curtis and Mara (1994) observed that the combination of sunlight, high dissolved oxygen concentration and pH is effective in removing faecal coliforms from facultative and maturation ponds. They suggested that humic substances which are ubiquitous in WSP absorb sunlight and pass the absorbed energy to oxygen (O_2), resulting in the formation of toxic forms of oxygen. This suggestion is supported by the observation that sunlight alone has little impact on the removal of faecal coliforms.

Curtis and Mara (1994) developed a model for faecal coliform removal (discounting any removal in the dark) as a function of sunlight, dissolved oxygen concentration and pH. Their proposed model is shown in equation 4.4, as follows:

$$\text{Log } K_{FC_T} = -2.76 + 0.000446 I_r + 0.323 \text{pH} + 0.0708 \text{DO} \qquad (4.4)$$

where

I_r = irradiance, ranging from 429 to 1096 Wm^{-2}

DO = concentration of dissolved oxygen, ranging from 0.8 to 7.5 mg/*l*

pH , ranging from 7.2 to 9.5

The researchers suggested that the application of the equation 4.4 should be used in areas that have a temperature range of 30–40^{o}C. This proposed temperature range is relatively higher than the average temperatures in warm climates areas and this raises questions about the applicability of the model in developing countries, which have annual average temperatures below 30^{o}C.

It has been proposed by Mara and Pearson (1998) that the design temperature of WSP should be taken as the mean temperature in the coldest month of the year. Therefore, using the temperature range proposed by equation 4.4 could lead to pond failure during cold seasons, when BOD removal in anaerobic and facultative pond will be very low. In addition, the application of equation 4.4 requires the designer to assume values of the variables of sunlight intensity, dissolved oxygen concentration and pH. Curtis and Mara (1994) provided the possible ranges of these variables to facilitate the use of their model. However, the proposed ranges are so wide that the accuracy of the model could be compromised. This could lead to unsafe design of the maturation ponds with possible risks to public health risk if unrestricted crop irrigation is to be practised.

4.4.5 Mara's empirical equation of faecal coliform removal in anaerobic pond

Marais' (1974) assumes that the faecal coliform die-off rate should be the same in anaerobic, facultative and maturation ponds. This assumption has drawn strong criticism from various researchers (Saqqar and Pescod, 1991; Sarikaya et al. 1987b; Mayo, 1995). It is argued that different environmental conditions, which are responsible for faecal coliform removal, prevail in each type of WSP. Sedimentation of solids-associated excreted pathogens is considered the major process by which excreted pathogens are removed from anaerobic ponds. Curtis and et al. (1992) have observed that the combination of sunlight intensity, oxygen and pH provide an effective mechanism for removing excreted pathogens from facultative and maturation ponds through a process of photo-oxidation.

Mara (2002) developed a faecal coliform removal model for anaerobic ponds based on data derived from a full-scale WSP in northeast Brazil. He observed that the removal of faecal coliforms from anaerobic ponds depended on the hydraulic retention time and temperature. He observed a satisfactory correlation between faecal coliform removal at temperature of 25°C and at a hydraulic retention time range of 0.8 - 6.8 days. The proposed faecal coliform die-off rate is shown in equation 4.5 as follows:

$$\frac{N_e}{N_i} = \frac{1}{(1 + K_{FC_T}\phi^{(T-20)}\theta_a)} \tag{4.5}$$

where

N_e = effluent faecal coliform concentration ($FC/100ml$)

N_i = influent faecal coliform concentration ($FC/100ml$)

K_{FC_T} = first order rate constant for faecal coliform removal (day^{-1}) = 2.0

ϕ = temperature coefficient of faecal coliform removal = 1.07

θ_a = hydraulic retention time in anaerobic pond (days)

T = mean air temperature (°C)

Modern design methods use Mara's empirical equation 4.5 to model faecal coliform removal in anaerobic ponds. The model suggests that faecal coliform removal is very effective if the hydraulic retention time and temperature increase. It is suggested that this results in greater sedimentation of solids-associated faecal coliform.

4.4.6 Von Sperling's faecal coliform removal model

Von Sperling (1999) developed a model for predicting faecal coliform removal in facultative and maturation ponds in Brazil. The ponds were located in different parts of Brazil with climates ranging from tropical to subtropical. The model was based on a dispersed hydraulic flow regime and had a satisfactory coefficient of correlation (R^2 = 0.847). The model was developed in terms of pond depth and hydraulic retention time. This approach appeared to be more satisfactory than other models that used environmental factors for which assumptions needed to be made. The empirical equation of Von Sperling (1999) to predict effluent faecal coliform concentration is shown in equations 4.6 and 4.7 as follows:

$$K_{FC_{20}} = 0.917H^{-0.877}\theta^{-0.329} \tag{4.6}$$

$$K_{FC_T} = K_{FC_{20}}\phi^{(T-20)} \tag{4.7}$$

where

K_{FC_T} = faecal coliform die-off rate at temperature T °C

$K_{FC_{20}}$ = faecal coliform die-off rate at 20°C

H = pond depth (m)

θ = mean hydraulic retention time (days)

ϕ = temperature coefficient of faecal coliform removal (= 1.07)

Von Sperling used his model to predict the effluent faecal coliform concentration in existing facultative and maturation ponds in Brazil. The model predicted accurately (R^2 = 0.959) the observed effluent faecal coliform concentration. This therefore validates the application of Von Sperling's empirical equation in designing maturation ponds for unrestricted crop irrigation.

Although Von Sperling's empirical equation 4.6 does not take account of the environmental factors of pH, dissolved oxygen concentration, algae and sunlight as proposed by other researchers, it is suggested that pond depth links directly with these environmental factors. Many researchers have found that shallow facultative and maturation ponds are far more effective in removing faecal coliforms than deeper ponds. They suggest that the full penetration of sunlight in shallow ponds enables the pond algae to photosynthesize intensely. This results in significant production of oxygen and a favourable pond pH, thereby facilitating the photo-oxidation process that kills faecal coliforms (Curtis and Mara, 1994). Deeper ponds, on the other hand, are inefficient in removing faecal coliforms because the sunlight cannot penetrate sufficiently well to encourage a high level of algal photosynthesis, with subsequently lower photo-oxidative activity.

Modern design methods use Von Sperling's empirical equation (equation 4.6) and the Arrhenius equation (equation 4.7) for modelling faecal coliform removal in facultative and maturation ponds. The design of maturation ponds is then based on a dispersed hydraulic flow regime using Von Sperling's empirical equation (equation 4.2) of the dispersion number model. The first-order equation of Wehner and Wilhelm (1956) is then used to predict the effluent faecal coliform concentration.

4.5 Modern design methods of maturation ponds

Modern methods for designing maturation ponds require the determination of the hydraulic retention time of the anaerobic and facultative ponds. The input design variables are treated as a range (Chapter 3) to accommodate their uncertainties. The determination of the random value of the hydraulic retention time for the anaerobic pond is calculated as follows:

$$\theta_a = \frac{L_i}{\lambda_v} \tag{4.8}$$

where
the design parameters L_i, and λ_v are random values selected from a proposed range in order to determine the random hydraulic retention time in anaerobic pond
θ_a = hydraulic retention time (days)
L_i = influent BOD concentration (mg/l)
λ_v = volumetric BOD loading (g/m^3 day)

The empirical equation of Von Sperling (1999) requires the determination of the hydraulic retention time of the facultative pond in order to model the faecal coliform removal from the pond. Modern design methods use the random values of the hydraulic retention time from the range of input design parameters. The procedures for calculating the random hydraulic retention time for the facultative pond are presented as follows:

$$\lambda_{sf} = 350(1.107 - 0.002T)^{T-25} \tag{4.9}$$

$$A_f = \frac{10 L_i Q_f}{\lambda_{sf}} \tag{4.10}$$

$$\theta_f = \frac{2 A_f H_f}{(2Q_f - 0.001 e A_f)} \tag{4.11}$$

where the design parameters T, L_i, Q_f, λ_{sf}, A_f and e are random values selected from a proposed range in order to determine the random hydraulic retention time (θ_f) of the facultative pond.
The subscript "f" refers to facultative pond

λ_{sf} = surface BOD loading (kg/ha/day)
T = temperature (^{o}C)
A_f= facultative pond area (m^2)
L_i = influent BOD concentration into facultative pond (mg/l)
Q_f = mean flow (m^3/day)
H_f = pond depth (m)
e = net evaporation (mm/day)

Modelling the faecal coliform removal in the first maturation pond requires the determination of its hydraulic retention time. The design of the first maturation pond is carried out as recommended by Mara (1997). The input design parameters are treated as a range. The computation of the random minimum hydraulic retention time in the first maturation pond is presented as follows:

$$\theta_{m1} = \frac{10 L_i H_{m1}}{0.75 \lambda_{sf}} \tag{4.12}$$

where the design parameters L_i, and λ_{sf}, are random values selected from a proposed range in order to determine the random hydraulic retention time (θ_{m1}) of the first maturation pond.
The subscript "*m1*" refers to first maturation pond
θ_{m1} = minimum hydraulic retention time in first maturation pond (days)
H_{m1}= design depth of the first maturation pond (m)
L_i = influent BOD concentration in first maturation pond (mg/l)
λ_{sf}= surface BOD loading in facultative pond (kg/ha day)
The design hydraulic retention time in the second and subsequent maturation ponds is based on a minimum retention time of 3−5 days as recommended by Marais (1974).

Modelling the faecal coliform removal from anaerobic ponds is carried out by applying the empirical equation of Mara (2002) as follows:

$$\left(N_e\right)_a = \frac{N_i}{(1 + K_{FC_T} \phi^{(T-20)} \theta_a)} \tag{4.13}$$

where
the design parameters N_i, θ_a, T, K_{FC_T} and ϕ are random values selected from proposed range,

$(N_e)_a$ = effluent faecal coliform concentration (per 100 ml)

N_i = influent faecal coliform concentration (per 100 ml)

K_{FC_T} = first-order rate constant for faecal coliform removal (day^{-1}) (= 2.0 at 20 ^{o}C)

ϕ = temperature coefficient for faecal coliform removal = 1.07

θ_a = hydraulic retention time (days)

T = air temperature (^{o}C)

Modelling the effluent faecal coliform concentration in the facultative pond is carried out by applying the equations of Von Sperling (1999) as follows:

$$d_f = \frac{1}{\left(\frac{L}{W}\right)_f} \tag{4.14}$$

$$K_{FC_{20}f} = 0.917 H_f^{-0.877} \theta_f^{-0.329} \tag{4.15}$$

$$K_{FC_{Tf}} = K_{FC_{20f}} \phi^{(T-20)} \tag{4.16}$$

$$(N_e)_f = (N_e)_a x \left(\frac{4a_f e^{\frac{1}{2d_f}}}{(1+a_f)^2 e^{\frac{a_f}{2d_f}} - (1-a_f)^2 e^{\frac{-a_f}{2d_f}}} \right) \tag{4.17}$$

$$a_f = \sqrt{(1 + 4K_{FC_Tf} \theta_f d_f})} \tag{4.18}$$

where

the subscript "*f*" refers to facultative pond;

the design variables, $(N_e)_f$, $(N_e)_a$, $K_{FC_T f}$, ϕ, $K_{FC_{20}f}$, d_f, θ_f and T are random values selected from their proposed range;

and:

$(N_e)_f$ = effluent faecal coliform concentration (per 100 ml)

$(N_e)_a$ = influent faecal coliform concentration (per100 ml)

$K_{FC_T f}$ = faecal coliform die-off rate at temperature T ^{o}C (day^{-1})

$K_{FC_{20} f}$ = faecal coliform die-off rate at 20^{o}C (day^{-1})

H_f = pond depth (m)

ϕ = temperature coefficient for faecal coliform removal = 1.07

d_f = dispersion numbers

L = pond length (m)

W = pond breath (m)

θ_f = hydraulic retention time (days)

T = air temperature (^{o}C)

Modelling the effluent faecal coliform concentration in the first maturation pond is carried out by applying the equation of Von Sperling (1999) as follows:

$$d_{m1} = \frac{1}{\left(\frac{L}{W}\right)_{m1}} \tag{4.19}$$

$$K_{FC_{20\,m1}} = 0.917 H_{m1}^{-0.877} \theta_{m1}^{-0.329} \tag{4.20}$$

$$K_{FC_{T_{m1}}} = K_{FC_{20_{m1}}} \phi^{(T-20)} \tag{4.21}$$

$$(N_e)_{m1} = (N_e)_f \, x \left(\frac{4 a_{m1} e^{\frac{1}{2 d_{m1}}}}{(1+a_{m1})^2 e^{\frac{a_{m1}}{2 d_{m1}}} - (1-a_{m1})^2 e^{\frac{-a_{m1}}{2 d_{m1}}}} \right) \tag{4.22}$$

$$a_{m1} = \sqrt{(1 + 4 K_{FC_T m1} \theta_{m1} d_{m1})} \tag{4.23}$$

where:

the subscript "*m1*" refers to first maturation pond

the design variables, $(N_e)_{m1}$, $(N_e)_f$, $K_{FC_{20m1}}$, ϕ, $K_{FC_{Tm1}}$, d_{m1}, θ_{m1} and T are random values selected from proposed range.

$(N_e)_{m1}$ = effluent faecal coliform concentration (per 100 ml)

$(N_e)_f$ = influent faecal coliform concentration (per 100 ml)

$K_{FC_{Tm1}}$ = faecal coliform die-off rate at temperature T ^{o}C (day^{-1})

$K_{FC_{20m1}}$ = faecal coliform die-off rate at 20^{o}C (day^{-1})

H_{m1} = pond depth (m)

ϕ = temperature coefficient for faecal coliform removal (= 1.07)

d_{m1} = dispersion number

L = pond length (m)

W = pond breath (m)

θ_{m1} = hydraulic retention time (days)

T = air temperature (^{o}C)

Unrestricted crop irrigation is satisfied by designing the second and subsequent maturation ponds to achieve an effluent faecal coliform concentration of less than 1000 FC per 100ml (WHO, 1989). The design procedure employs iteration method. The empirical equations of Von Sperling (1999) are used to model faecal coliform removal. The design procedure is presented as follows:

$$d_m = \frac{1}{\left(\frac{L}{W}\right)_m} \tag{4.24}$$

$$K_{FC_{20m}} = 0.917 H_m^{-0.877} \theta_m^{-0.329} \tag{4.25}$$

$$K_{FC_T m} = K_{FC_{20m}} \phi^{(T-20)} \tag{4.26}$$

$$(N_e)_m = (N_e)_{m1} x \left(\frac{4 a_m e^{\frac{1}{2d_m}}}{(1+a_m)^2 e^{\frac{a_m}{2d_m}} - (1-a_m)^2 e^{\frac{-a_m}{2d_m}}} \right)^n \tag{4.27}$$

$$a_m = \sqrt{(1 + 4K_{FC_T m} \theta_m d_m)} \tag{4.28}$$

where

The subscript "*m*" refers to second and subsequent maturation ponds

The design variables, $(N_e)_m$, $(N_e)_{m1}$, $K_{FC_{Tm}}$, $K_{FC_{20m}}$, ϕ, d_m, θ_m and T are random values selected from proposed range.

$(N_e)_m$ = effluent faecal coliform concentration (per 100 ml)

$(N_e)_{m1}$ = influent faecal coliform concentration (per 100 ml)

$K_{FC_{Tm}}$ = faecal coliform die-off rate at temperature T ^{o}C (day^{-1})

$K_{FC_{20m}}$ = faecal coliform die-off rate at 20^{o}C (day^{-1})

H_m = pond depth (m)

ϕ = temperature coefficient for faecal coliform removal (= 1.07)

d_m = dispersion numbers

L = pond length (m)

W = pond breath (m)

θ_m = hydraulic retention time (= 3–5 days)

T = air temperature (^{o}C)

n = number of second and subsequent maturation ponds

4.6 Uncertainty of input design variables and coefficients in Von Sperling's and Mara's empirical equations

The empirical equations of Von Sperling (1999) and Mara (2002) rely upon coefficients and design variables which are subjected to uncertainty when modelling faecal coliform removal. The design variables that are used in these models are temperature, dispersion numbers and the hydraulic retention time, while the coefficient used in these models is the temperature coefficient for faecal coliform removal. These design variables and the temperature coefficients are known to vary in practice and cannot be determined with a high level of accuracy.

Various researchers have proposed different values for the temperature coefficient in predicting faecal coliform removal in WSP. Table 4.1 shows the variation of proposed temperature coefficients found in the literature relating to the modelling of faecal coliform removal.

Table 4.1 Temperature coefficient variation

Temperature coefficient (ϕ)	Reference
1.19	Marais (1974)
1.06	Sherry and Parker (1979)
1.17	Mills et al. (1992)
1.07	Yanez (1993)
1.08	Mayo (1995)

Table 4.1 suggest that the temperature coefficient used in the empirical equations of Von Sperling (1999) and Mara (2002) cannot be determined accurately during the design stages of WSP. The coefficient is subjected to uncertainty and varies within a certain range. This suggests that modelling of faecal coliform removal in anaerobic, facultative and maturation ponds should not be based on deterministic average single values because of the variation of the coefficients. Design of maturation ponds in this way could lead to an overestimation of faecal coliform removal, making unrestricted crop irrigation potentially unsafe.

Dispersion number is one of the design variables used in the empirical equation of Von Sperling (1999) for modelling the dispersion numbers of faecal coliform in facultative and maturation ponds. Dispersion numbers have been known to vary in practice. The natural phenomena of wind mixing and temperature are responsible for dispersion in WSP and the degree of dispersion produced by these factors depends on the extent of wind mixing and temperature changes. Dispersion numbers may also vary due to fluctuation of the momentum of the influent wastewater flow.

Von Sperling (2002) compared the accuracy of the four different models proposed by Von Sperling (1999), Yanez (1993), Agunwamba et al. (1992) and Polprasert and Bhattarai (1985) in predicting the dispersion numbers in WSP. He found that the four proposed models predicted different values. Various researchers have observed different values of dispersion numbers in full-scale and pilot-scale WSP, as shown in Table 4.2.

Table 4.2 Dispersion numbers

Dispersion number (d)	Reference
0.0625 - 4	Thirumurthi (1969)
0.395 – 1.71	Finney and Middlebrooks (1980)
0.115 – 0.215	Polprasert and Bhattarai (1985)
0.110 – 0.213 0.153 – 0.189	Agunwamba et al.(1992) − tracer Agunwamba et al.(1992) – model
0.371 – 0.595	Marecos do Monte and Mara (1987)

Table 4.2 shows that dispersion numbers vary significantly in WSP. Use of the dispersion number model of Von Sperling (1999) to predict faecal coliform removal in WSP based on deterministic single average values of dispersion numbers could therefore lead to an unrealistic and unsafe design of maturation ponds. The best approach to manage the uncertainty of dispersion numbers when modelling faecal coliform removal is to treat the dispersion numbers as a range.

The empirical equations of Von Sperling (1999) and Mara (2002) to predict faecal coliform removal include the design variables of influent faecal coliform concentration, temperature and hydraulic retention time. Chapter 3 has demonstrated that these parameters vary in practice and are subjected to uncertainty.

Modern design methods recognize the weakness of treating design variables as deterministic average single values. Modern methods therefore treat the input design variables and the coefficients used in models as a range, which takes into account the extent of any uncertainty. The average values of the input design variables that the designer is confident with, are used as a basis to establish the lower and upper values of the proposed range. Monte Carlo simulation is used to select randomly any design variable value within the proposed range of the input variables.

4.7 Monte Carlo Simulation

Von Sperling (1996) suggested that Monte Carlo simulation should be used when designing WSP because it is an efficient way to manage the uncertainty of the input design variables and coefficients. Modern design methods use Monte Carlo simulation in designing anaerobic, facultative and maturation ponds for unrestricted crop irrigation. Monte Carlo simulation works by selecting at random a value of each input design parameter and the coefficient of the models within a specified range. The Monte Carlo simulation then determines the design of WSP for this selection of the parameter values. This random value design procedure is repeated for any required number of times. Finally, the design outputs of pond area, hydraulic retention time and effluent faecal coliform concentration are statistically analysed to produce a frequency histogram and cumulative frequency curves. The final design outputs (pond area, hydraulic retention time and effluent faecal coliform concentration) are then chosen according to either a 95%ile or 50%ile basis depending on the project's budget and the acceptable health risk.

4.7.1 Definition of the input design range

Monte Carlo simulation uses a uniform probability distribution to generate a range of the input design values. Uniform probability distribution is suitable for design parameters that contain end values of the proposed range. This type of distribution is especially suited to the design of WSP because average deterministic values of input design parameters are known with high certainty.

Vose (1996) suggested that the cumulative probability distribution function for a uniform distribution of any range that has known end values could be expressed as equation 4.29:

$$F(x) = \frac{x - A}{B - A} \tag{4.29}$$

where

x = any random input design value within a range

A = the lower input design value of a range

B = the upper input design value of a range

Monte Carlo simulation utilizes the inverse function of the cumulative density function, which according to Vose (1996) is given in equation 4.30 as follows:

$$F^{-1}(x_i) = A + (B - A)\upsilon_i \tag{4.30}$$

$$x_i = A + (B - A)\upsilon_i$$

where

$\upsilon_i =$ any random number value (0 - 1)

A random number is any natural number that is found between a range of 0 and 1. Monte Carlo simulation uses the random number generated by the computer to compute the random input design value within the proposed range. Monte Carlo simulation uses a random design value (x_i), given by equation 4.30 from the input design variables, to design pond area, hydraulic retention time and the effluent faecal coliform concentration. Depending on the number of the design calculations required, the Monte Carlo simulation generates similar input random design values.

In summary, this chapter has shown that a dispersed hydraulic flow regime is the realistic flow pattern that occurs in WSP. The chapter has presented the empirical equation of Mara (2002) for modelling faecal coliform removal in anaerobic ponds. The empirical equations of Von Sperling (1999) have been presented and it has been shown how they are used to design maturation ponds for unrestricted crop irrigation. The equations of Von Sperling (1999) and Mara (2002) have been shown to include design variables and coefficients that are subjected to uncertainty. The chapter has shown that modern design methods for WSP treat the input design variables as a range, and how these modern methods manage these uncertainties by using Monte Carlo simulations.

In conclusion, modern design methods represent the rational procedures for designing WSP. Modern methods are based on modelling a realistic dispersed hydraulic flow regime and incorporate data from sound research. These design procedures could lead to effluent from WSP that is safe to use for unrestricted crop irrigation, and WSP systems that use the available land and funds economically.

CHAPTER 5

THE VISUAL BASIC COMPUTER PROGRAM FOR THE MODERN DESIGN OF WSP

5.1 Introduction

Modern design methods for WSP provide an efficient way of managing inherent uncertainties in various input parameters through the use of Monte Carlo simulations (Chapter 4). They achieve this by treating the input design parameters as a range rather than as single values. The design calculations are carried out by selecting randomly the input design variables within the proposed range. The input design parameters are expressed as a uniform distribution range.

The inverse cumulative function of the uniform probability distribution is one of the fundamental aspects of the Visual Basic computer program that has been developed. In addition, Monte Carlo simulation processes have been incorporated into the program. Simulations are defined mathematically as the number of times the design calculations are repeated. In this way the Monte Carlo simulations provide random output design data that are statistically analysed.

The Visual Basic computer program has incorporated Monte Carlo simulations to design anaerobic, facultative and maturation ponds. Each run of a simulation provides random design values of the hydraulic retention times of anaerobic pond, facultative pond and the first maturation pond, which are used in the empirical equations of Mara (2002) and Von Sperling (1999) for modelling faecal coliform removal.

The computer program has incorporated the equations developed by Mara and Pearson (1986) and Mara et al. (1997a) for designing anaerobic ponds. The recommended ranges of the volumetric organic loading rate and BOD removal in anaerobic ponds have been manipulated to follow the pattern depicted by temperature variations. The program has also used the equation of Mara (1987) for designing facultative ponds. The surface BOD loading rate has been modelled to follow the temperature variations. Finally, the Visual Basic program has incorporated the equations of Mara'(2002) and Von Sperling (1999) for designing the second and subsequent maturation ponds.

Visual Basic programming language is a programming language that has been developed by Microsoft Inc. to facilitate computer programming for design in industry. The program code consists of written instructions, which command the computer to perform the tasks defined by the user. The Visual Basic programming language is user-friendly due to its compatibility with Excel Spreadsheet. New users of the Visual Basic program developed here, who have basic knowledge of Excel Spreadsheet, should be able to run the program without any difficulties.

The input design parameters for a given WSP system are entered in Excel Spreadsheet rather than in the program module, as is the case with most existing programming languages. The computer program can access these input design parameters in Excel Spreadsheet. In addition, the computer program presents the output design data in Excel Spreadsheet. This is useful because it enables the statistical tools found in Excel Spreadsheet to produce frequency cumulative curves and histograms.

5.2 Generation of random design values from the input design parameters in the Visual Basic program

The input design parameters for the WSP are expressed as a range in the Visual Basic program. The range is defined by the uniform probability distribution. The end values of the range are established by the designer, based on the average deterministic single value of the input design value, which is known with high certainty. The uniform probability distribution function for the input design range is defined in equation 4.29 as follows:

$$F(x) = \frac{x - A}{B - A}$$

where

x = any random input design value within a range

A = the lower input design value of a range

B = the upper input design value of a range

The Visual Basic program utilizes the inverse function of the uniform distribution range in order to compute the random design value of the input parameters. The equation is presented in equation 4.30 as follows:

$$F^{-1}(x_i) = A + (B - A)\upsilon_i$$

$$x_i = A + (B - A)\upsilon_i$$

where

$\upsilon_i =$ any random number value (0–1)

In order to compute the random design value of the input range, the Visual Basic program executes a random number function that generates random numbers ranging between 0 and 1. The program selects randomly the random numbers such that the computation of the random design value of the input design parameters is effected during each run of the simulation.

5.3 Application of modern design methods in Visual Basic program

5.3.1 Input range of design parameters

The first design step in the development of the Visual Basic computer program is the manipulation of the input range of the design parameters. The input design parameters are entered in Excel Spreadsheet in a form of end values such that the computer program can access them. The designer establishes the end values of a range for every input design parameter based on the level of its uncertainty. In most cases, the average deterministic single value of the input design parameter is used to determine the range of the parameter. It is suggested by Von Sperling (1996) that average values of the input design parameters as used in traditional design methods should be used in establishing the range of the input design parameters.

Von Sperling (1996) recommends that the lower and upper design values of the proposed range be determined by assuming a percentage value, which reflects the level of uncertainty of the average deterministic single value. The procedure of determining the input range of the design parameters is presented in equations 5.1 and 5.2 as follows:

$$X_{\min} = \overline{X} - a\overline{X} \tag{5.1}$$

$$X_{\max} = \overline{X} + a\overline{X} \tag{5.2}$$

where

X_{min} = lower end value of the input design range

X_{max} = upper end value of the input design range

$\overline{X}$ = average value of the input design parameter

a = any assumed percentage value based on the level of the uncertainty

The input design range of temperature should be determined by using the daily mean temperature in the coldest month as the lower end value of the range and the daily mean temperature in the hottest month as the upper end value of the range.

The following is the list of the input design parameters that have been used in the Visual Basic computer program for the modern design of WSP:

- Per capita BOD contribution (g BOD per person per day). This input design value varies from 30 to 70 g per person per day in developing countries (Mara et al. 1992). A suitable average single value of 40 g per person per day can be used in equations 5.1 and 5.2 for establishing a range of the per capita BOD contribution.

- Per capita wastewater production (*l* per person per day). This input design value is used to determine the BOD concentration (mg/*l*) in raw wastewater. Von Sperling (1996) suggests that an average wastewater flow rate of 120*l* per person per day is appropriate in developing countries. This deterministic average single value is used in equations 5.1 and 5.2 to determine the input range of the per capita wastewater production.

- Design population. This input design parameter is needed so that the design wastewater flow rate from the served community can be estimated. It is used in conjunction with the per capita wastewater flow rate. The design population is established by using equation 5.3 as follows:
-

$$P_d = P_O(1+r)^n \tag{5.3}$$

where
P_d = design population of the served community
P_o = initial population of the served community
r = population growth rate
n = design period of WSP

The design population is used in equations 5.1 and 5.2 to determine the input range of the population. The values of the population range are then used in equation 5.4 for establishing the design flow range for WSP as follows:

$$Q = \frac{P_d q}{1000} \tag{5.4}$$

where

Q = design flow rate (m^3/day)

P_d = design population

q = per capita wastewater production (l per person per day)

- Net evaporation rate (mm/day). The lower end value of the proposed range should be the minimum net evaporation rate in the coldest month. The upper end value of the proposed range should be obtained from the hottest month.

- Depth of the anaerobic pond (m). This input design parameter is considered constant. The lower and upper end values of the proposed depth range are assigned equivalent values. A pond depth of 4 m is normally used to design an anaerobic pond such that the available land is used economically.

- Depth of the facultative pond (m). This input design parameter is considered constant. The lower and upper end values of the proposed depth range are assigned equivalent values. A shallow depth of 1.5 m is normally used to design a facultative pond (Mara and Pearson, 1998).

- Depth of the maturation ponds (m). This input design parameter is considered constant. The lower and upper end values of the proposed depth range are assigned equivalent values. A shallow depth of 1.0 m is normally used in designing maturation ponds (Mara and Pearson, 1998).

- Temperature (^{o}C). The input design range of temperature is established by assigning the mean daily temperatures in the coldest month as the lower end value of the proposed temperature range. The upper value of the temperature range should be established from the mean daily temperatures in the hottest month.

- Influent faecal coliform concentration (FC per 100 ml). This input design parameter influences the number of maturation ponds required. The maturation ponds are designed to satisfy the practice of the unrestricted crop irrigation. Mara et al. (1992) have proposed that raw wastewater contains faecal coliforms in concentrations ranging from 10^7 to 10^9 FC per 100ml. They suggested that an average design value of 10^8 FC per 100ml is appropriate in designing maturation ponds. This proposed single average value is used in equations 5.1 and 5.2 to establish the lower and upper end values of the faecal coliform concentration range.

- First-order constant rate for faecal coliform removal in an anaerobic pond (day^{-1}). Mara (2002) suggests an average design value of the first-order constant rate for faecal coliform removal in anaerobic ponds as 2. This deterministic single average value is used in equations 5.1 and 5.2 to establish the lower and upper end values of the input design range.

- Temperature coefficient for faecal coliform removal. This is a dimensionless input design parameter, which is used in the Arrhenius-type equation. Von Sperling (1999) suggests an average design value of 1.07 be used for modelling faecal coliform in WSP. This deterministic average single value is used in equations 5.1 and 5.2 to establish the lower and upper end values of the proposed range.

- The minimum design hydraulic retention time in the second and subsequent maturation ponds. This input design parameter is used in the empirical equations of Von Sperling (1999) for modelling faecal coliform removal in the second and subsequent maturation ponds. The modern design of maturation ponds is based on iterations and 95-%ile of the effluent faecal coliform concentration. Marais (1974) proposed a minimum design range for the hydraulic retention times in maturation ponds as 3–5 days. This input design range is used in the Visual Basic program for designing the second and the subsequent maturation ponds.

- Dispersion numbers in a facultative pond. This is one of the input design parameters that is used in the Visual Basic computer program for modelling faecal coliform removal in a facultative pond. Von Sperling (1999) has suggested that dispersion numbers are the reciprocal of the ratio of the dimensions of facultative pond (length to width ratio). Mara et al. (1992) have suggested that facultative ponds should have a length-to-width ratio of up to 10:1. Therefore, the average single value of the dispersion numbers in facultative ponds can be taken as 0.1. This deterministic average value is used in equations 5.1 and 5.2 to establish the lower and upper end values of the range of dispersion numbers in a facultative pond.

- Dispersion numbers in maturation ponds. The determination of the average dispersion numbers in maturation ponds uses the empirical equation of Von Sperling (1999), which is given as a reciprocal of the ratio of the maturation pond dimensions (length-to-width ratio). Maturation ponds are designed to have a higher length-to-width than facultative ponds to facilitate effective faecal coliform removal (Shilton and Harrison, 2003). For the purposes of illustration, a length-to-width ratio of 20:1 is used to model faecal coliform removal in maturation ponds. Therefore, the average dispersion number of 0.05 can be used in equations 5.1 and 5.2 to establish the lower and upper end values of the range of the dispersion numbers.

- The minimum hydraulic retention time in anaerobic ponds. This input design parameter is considered constant. Mara (1997) suggests a 1-day retention time is sufficient for effective BOD removal in anaerobic ponds. The Visual Basic program assigns the random design value of hydraulic retention in an anaerobic pond as 1 day if the computed random retention time is less than 1 day. This 1-day retention time is then used to design anaerobic ponds and the modelling of faecal coliform removal in the empirical equation of Mara (2002).

- The minimum hydraulic retention time in facultative ponds. This input design parameter is considered as a constant value. Mara (1997) suggests 4 days hydraulic retention time as the minimum in a facultative pond.

- The minimum hydraulic retention time in maturation ponds. This input design parameter is considered as a constant value. Marais (1974) has suggested that 3 days retention time is adequate to minimize short-circuiting in maturation ponds. The Visual Basic program uses 3 days as the minimum retention time in designing maturation ponds.

- Number of simulations. This input design parameter is entered as a constant value in the computer program. It indicates the number of times the design calculations are to be repeated.

These suggested input design parameters are entered by the designer in the Excel Spreadsheet to incorporate the degree of the uncertainty of the single values of average input parameters. The Visual Basic program uses an inverse cumulative probability uniform function, which computes the random design values of the input design parameters according to equation 4.30.

The computation of the random design values is aided by the random number function, which generates a random number between 0 and 1 for every run of the simulation.

5.3.2 Monte Carlo simulations for the design of WSP

The computer program has been developed in such way that the design calculations for anaerobic, facultative and first maturation ponds are carried out with a number of simulations. At every run of a simulation, new sets of random design values are selected by the computer program within the defined input design range until the final simulation run.

The design of the second and subsequent maturation ponds use the random design values of the hydraulic retention time selected from the minimum range of 3–5 days. These random values are used in the empirical equation of Von Sperling (1999) to model faecal coliform removal in maturation ponds. Figure 5.1 presents the procedure for the Monte Carlo simulation. The Visual Basic computer program that has been developed is based on this procedure.

The logical part of the program is based on the application of the random design values of temperature when determining the volumetric organic loading rate and BOD removal in anaerobic and facultative ponds. The computer program determines the random design values of the volumetric organic loading rate and BOD removal for every run of a simulation depending on the random value for temperature that has been selected. In addition, the computer program determines the random design value of overall BOD removal in anaerobic and facultative ponds when designing the first maturation pond depending on the selected random temperature.

The program uses four temperature conditions, which must at least be satisfied for every selected random temperature. This results in a logical selection of volumetric organic loading and BOD removal in anaerobic and facultative ponds such that they can mimic the pattern depicted by the temperature variation. This enables the design of the facultative and first maturation ponds to be based on random values of their influent BOD.

At every run of a simulation, the computer program selects randomly the temperature from the proposed range and the selected temperature is compared with the four temperature conditions as suggested by Mara and Pearson (1986) and Mara et al. (1997a). The first temperature condition is satisfied when the selected random temperature (T) <10^{o}C; the

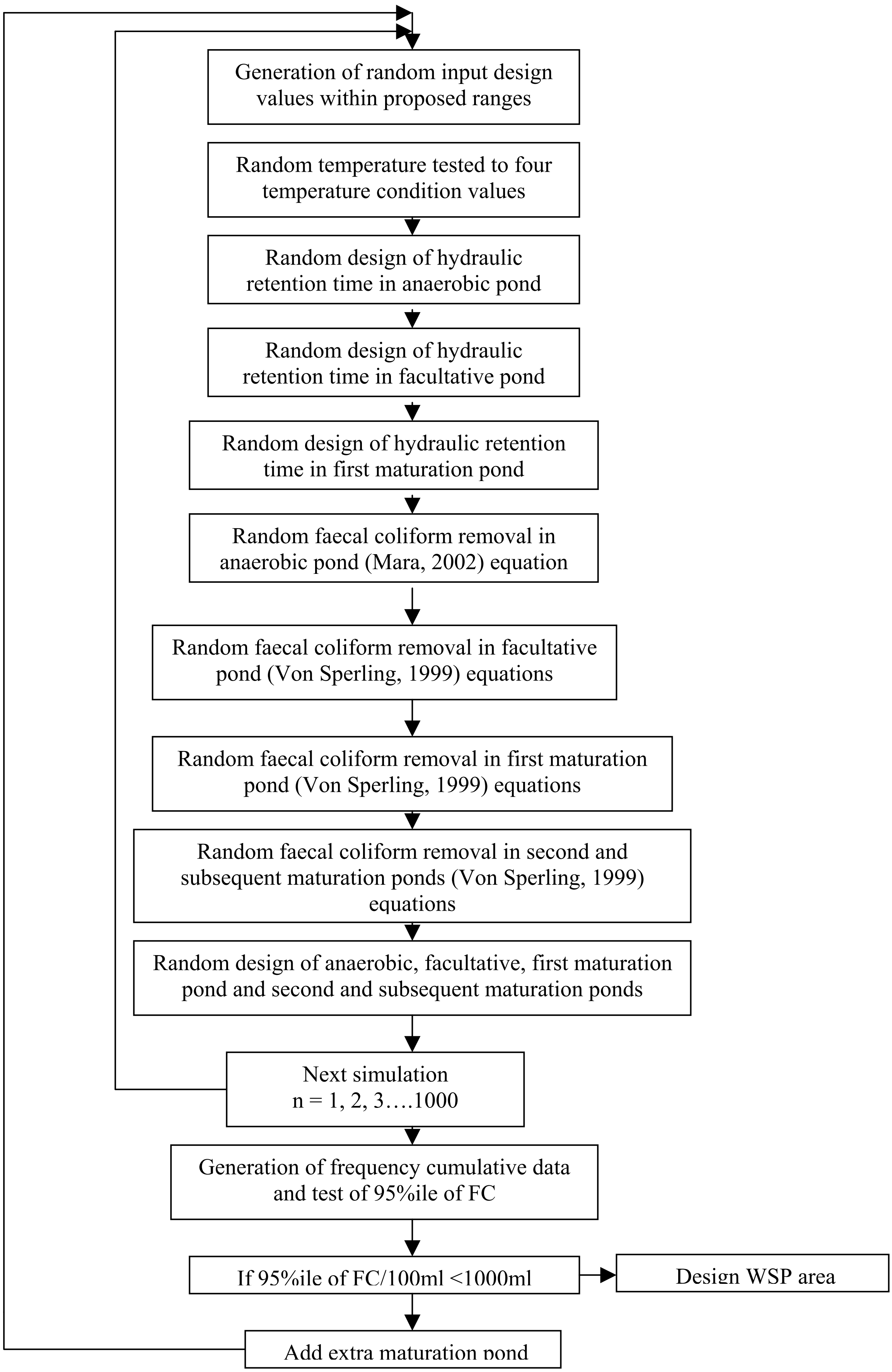

Figure 5.1 Procedure for the Monte Carlo simulation

computer program computes equations 5.5 and 5.6, and assigns the computed random values to these design variables:

$$\lambda_v = 100$$

$$\left(L_i\right)_f = 0.6\left(L_i\right)_a \tag{5.5}$$

$$\left(L_i\right)_{m1} = 0.3\left(L_i\right)_a \tag{5.6}$$

where

λ_v = volumetric organic loading rate (g/m^3 day)

$(L_i)_a$ = random design value of influent BOD in anaerobic pond (mg/l)

$(L_i)_f$ = influent BOD in facultative pond (mg/l)

$(L_i)_{m1}$ = influent BOD into first maturation pond (mg/l)

The selected random temperature, the volumetric organic loading rate and the random influent BOD concentration in the facultative and first maturation ponds are then used in designing the random hydraulic retention time in the anaerobic, facultative and first maturation ponds.

The second temperature condition is satisfied when the selected random temperature is between 10 and 20^{o}C; the computer program computes equations 5.7, 5.8 and 5.9, and assigns the computed random values to these design variables:

$$\lambda_v = 20T - 100 \tag{5.7}$$

$$\left(L_i\right)_f = \frac{100 - \left(2T + 20\right)}{100} x\left(L_i\right)_a \tag{5.8}$$

$$\left(L_i\right)_{m1} = 0.3\left(L_i\right)_a \tag{5.9}$$

where

T and $(L_i)_a$ are random design parameters

The selected random temperature, random volumetric organic loading rate and the random influent BOD concentration in the facultative and first maturation ponds are then used to design the random hydraulic retention time in the anaerobic, facultative and first maturation ponds.

The third temperature condition is satisfied when the selected random temperature is between 20 and 25°C; the computer program computes equations 5.10, 5.11 and 5.12, and assigns the computed random values to these design variables:

$$\lambda_v = 10T + 100 \tag{5.10}$$

$$(L_i)_f = \frac{100 - (2T + 20)}{100} x (L_i)_a \tag{5.11}$$

$$(L_i)_{m1} = 0.2(L_i)_a \tag{5.12}$$

where

T and $(L_i)_a$ are random design parameters

The selected random temperature, random volumetric organic loading rate and the random influent BOD concentration in the facultative and first maturation ponds are then used to design the random hydraulic retention time in the anaerobic, facultative and first maturation ponds.

The fourth temperature condition is satisfied when the selected random temperature is above 25°C; the computer program computes equations 5.13 and 5.14, and assigns the computed random values to these design variables:

$$\lambda_v = 350$$

$$(L_i)_f = 0.3(L_i)_a \tag{5.13}$$

$$(L_i)_{m1} = 0.2(L_i)_a \qquad 5.14)$$

where

$(L_i)_a$ is a random design parameter

The selected random temperature, the volumetric organic loading rate and the random influent BOD concentration in the facultative and first maturation ponds are then used to design the random hydraulic retention time in the anaerobic, facultative and first maturation ponds.

5.3.2.1 Random design computation of the hydraulic retention time in anaerobic ponds

The developed program uses Monte Carlo simulations to design the random hydraulic retention time in anaerobic pond for every run of a simulation. The random design of the hydraulic retention time in anaerobic ponds is presented in equation 4.8 as follows:

$$\theta_a = \frac{(L_i)_a}{\lambda_v}$$

where
θ_a = hydraulic retention time (days)
$(L_i)_a$ = influent BOD concentration in anaerobic pond (mg/l)
λ_v = volumetric BOD loading (g/m^3 day)

The influent BOD concentration $(L_i)_{a,}$ is randomly selected from the proposed uniform distribution range. The volumetric organic loading rate (λ_v) is obtained from any of the four defined temperature conditions which satisfy the selected random temperature. For the next run of the simulation, a new set of the random values of the influent BOD concentration $(L_i)_{a,}$ and volumetric organic loading ($\lambda_{v},$) are selected for the random design of the hydraulic retention time (θ_a).

The computer program compares the computed random design value of the hydraulic retention time (θ_a) with the minimum hydraulic retention time of 1 day. If the computed random retention time is less than 1 day, the computer program assigns a value of 1 day as the random hydraulic retention time in the anaerobic pond for that simulation.

5.3.2.2 Random design computation of the hydraulic retention time in facultative pond

Monte Carlo simulation is used by the computer program to compute the random design value of the hydraulic retention time in the facultative pond. The computer program selects randomly the input design parameters of temperature, design flow, net evaporation rate from their proposed uniform distribution range. The random value of the surface BOD loading rate is determined from the equation developed by Mara (1987), which is a function of the selected random temperature. The random design value of the influent BOD concentration is obtained from the four defined temperature conditions. The program determines the random design value of the hydraulic retention time in facultative ponds as presented in equations 4.9, 4.10 and 4.11 as follows:

$$\lambda_{sf} = 350(1.107 - 0.002T)^{T-25}$$

$$A_f = \frac{10(L_i)_f Q_f}{\lambda_{sf}}$$

$$\theta_f = \frac{2A_f H_f}{(2Q_f - 0.001eA_f)}$$

where

the subscript "f" refers to facultative pond

the parameters T, $(L_i)_f$, Q_f, λ_{sf}, A_f and e are random design values

θ_f = hydraulic retention time in facultative pond (days)

λ_{sf} = surface BOD loading (kg/ha day)

T = temperature (^{o}C)

A_f = facultative pond area (m^2)

$(L_i)_f$ = influent BOD concentration in the facultative pond (mg/l)

Q_f = mean flow (m^3/day)

H_f = pond depth (m)

e = net evaporation (mm/day)

The computed random design value of the hydraulic retention time (θ_f) is compared with the recommended minimum hydraulic retention of 4 days in facultative pond. If the computed random retention time is less than 4 days, the computer program assigns a random design value of 4 days as the random hydraulic retention time in the facultative pond for that simulation.

For the next run of the simulation, new sets of the random design values of temperature T, design flow Q_f, net evaporation e, and influent BOD concentration $(L_i)_f$, are selected from their proposed uniform distribution range. These random design values are used to compute the next random design value of the hydraulic retention time in the facultative pond.

5.3.2.3 Random design computation of the hydraulic retention time in first maturation ponds

Monte Carlo simulation is used by the computer program to determine the random design value of the hydraulic retention time in the first maturation pond. The random design value of the surface BOD loading rate is determined from the equation developed by Mara (1987), which is a function of the selected random temperature. The random design value of the influent BOD concentration is obtained from the four defined temperature conditions. The random design hydraulic retention time of the first maturation pond is carried out by employing the equation recommended by Mara (1997). The program computes the random design value of the hydraulic retention time in the first maturation pond as presented in equations 4.9, and 4.12 as follows:

$$\lambda_{sf} = 350(1.107 - 0.002T)^{T-25}$$

$$\theta_{m1} = \frac{10(L_i)_{m1} H_{m1}}{0.75\lambda_{sf}}$$

where

the subscript "*m1*" refers to first maturation pond

the parameters T, $(L_i)_{m1}$ and λ_{sf}, are random design values

θ_{m1} = minimum hydraulic retention time in first maturation pond (days)

H_{m1}= design depth of the first maturation pond (m)

$(L_i)_{m1}$ = influent BOD concentration in first maturation pond (mg/*l*)

λ_{sf}= surface BOD loading in facultative pond (kg/ha day)

The computed random design value of the hydraulic retention time (θ_{m1}) is compared with the minimum hydraulic retention time of 3 days. If the computed random value is less than 3 days, the computer program assigns a value of 3 days as the random hydraulic retention time in the first maturation pond. In addition, the computed random value of the hydraulic retention time in first maturation pond is again compared with the hydraulic retention time in the facultative pond. If the random design value of the hydraulic retention time in the first maturation pond is more than the computed value of the hydraulic retention time in the facultative pond, the computer program assigns the computed random value of the first maturation pond as that of the facultative pond.

For the next run of the simulation, new sets of the random design values of temperature (T) and influent BOD concentration $(L_i)_{m1}$, are selected from the proposed uniform distribution range. These new selected sets of random design values are used to compute the next random design of the hydraulic retention time (θ_{m1}) in the first maturation pond.

5.3.2.4 Random design computation of the hydraulic retention time in the second and subsequent maturation ponds

The random design values of the hydraulic retention time in the second and subsequent maturation ponds is selected from the minimum retention time range of 3–5 days, as recommended by Marais (1974). The computer program uses Monte Carlo simulations to select randomly the design value of the hydraulic retention time from the proposed uniform distribution range. These random values of the hydraulic retention time are used in the empirical equation of Von Sperling (1999) for modelling faecal coliform removal. For the next run of the simulation, the computer program selects a new set of random design values for the hydraulic retention time. These new values are used to model faecal coliform removal in the second and subsequent maturation ponds.

5.3.2.5 Random design computation of the effluent faecal coliform concentration in anaerobic ponds using the empirical equation of Mara (2002)

Monte Carlo simulation is used by the computer program to compute the random design value of the effluent faecal coliform concentration in anaerobic ponds. The computer program selects randomly the input design variables of temperature T, influent faecal coliform concentration $(N_i)_a$, the first-order rate constant for faecal coliform removal K_{FC_T} , and the temperature coefficient of faecal coliform removal ϕ , from their proposed uniform distribution range. The random design value of the hydraulic retention time θ_a, is determined from equation 4.8, which has been observed to be a function of random values of the volumetric organic loading rate λ_V, and the influent BOD concentration $(L_i)_a$. The computer program determines the random design value of the effluent faecal coliform concentration in anaerobic ponds by employing the empirical equation of Mara (2002), as presented in equation 4.13 as follows:

$$\left(N_e\right)_a = \frac{\left(N_i\right)_a}{(1+K_{FC_T}\phi^{(T-20)}\theta_a)}$$

where

the subscript "*a*" refers to anaerobic pond

$(N_e)_a$ = effluent faecal coliform concentration (per 100 ml)

N_i = influent faecal coliform concentration (per 100 ml)

K_{FC_T} = first order rate constant for faecal coliform removal (day^{-1})

ϕ = temperature coefficient of faecal coliform removal

θ_a = hydraulic retention time (days)

T = air temperature (^{o}C)

The next run of the simulation selects new sets of random design values of temperature T, influent faecal coliform concentration $(N_i)_a$, the first-order rate constant for faecal coliform removal K_{FC_T}, temperature coefficient of faecal coliform removal ϕ and the hydraulic retention time θ_a, from the uniform distribution range.

5.3.2.6 Random design computation of the effluent faecal coliform concentration in facultative pond using the empirical equations of Von Sperling'(1999)

The computer program uses Monte Carlo simulations to compute the random design value of the effluent faecal coliform concentration in facultative pond. The program selects randomly the input design variables of temperature T, influent faecal coliform concentration $(N_i)_f$, the temperature coefficient of faecal coliform removal ϕ and dispersion numbers d_f, from their proposed uniform distribution range. The random design value of the hydraulic retention time θ_f, is determined from equation 4.11. The computer program determines the random design value of the effluent faecal coliform concentration in facultative pond by from the empirical equations of Von Sperling (1999), as presented in equations 4.14, 4.15, 4.16, 4.17 and 4.18 as follows:

$$d_f = \frac{1}{\left(\frac{L}{W}\right)_f}$$

$$K_{FC_{20}f} = 0.917H_f^{-0.877}\theta_f^{-0.329}$$

$$K_{FC_{Tf}} = K_{FC_{20f}}\phi^{(T-20)}$$

$$\left(N_e\right)_f = \left(N_e\right)_a x \left(\frac{4a_f e^{\frac{1}{2d_f}}}{\left(1+a_f\right)^2 e^{\frac{a_f}{2d_f}} - \left(1-a_f\right)^2 e^{\frac{-a_f}{2d_f}}} \right)$$

$$a_f = \sqrt{(1+4K_{FC_T f}\theta_f d_f)}$$

where

the subscript "*f*" refers to facultative pond

$(N_e)_f$ = effluent faecal coliform concentration (per 100 ml)

$(N_e)_a$ = influent faecal coliform concentration (per 100 ml)

$K_{FC_T f}$ = faecal coliform die-off rate at temperature T °C

$K_{FC_{20} f}$ = faecal coliform die-off rate at 20°C

H_f = pond depth (m)

ϕ = temperature coefficient for faecal coliform removal = 1.07

d_f = dispersion numbers

L = pond length (m)

W = pond breath (m)

θ_f = hydraulic retention time (days)

T = air temperature (°C)

For the next run of the simulation, the computer program selects new sets of the random design values of temperature T, influent faecal coliform concentration $(N_e)_a$, temperature coefficient of faecal coliform removal ϕ, dispersion numbers d_f, and the hydraulic retention time θ_f, from their proposed uniform distribution range.

5.3.2.7 Random design computation of the effluent faecal coliform concentration in first maturation pond using the empirical equations of Von Sperling (1999)

The computer program uses Monte Carlo simulations to compute the random design value of the effluent faecal coliform concentration in the first maturation pond. The computer program selects randomly the input design variables of temperature T, influent faecal coliform concentration $(N_e)_f$, temperature coefficient of faecal coliform removal ϕ, and dispersion numbers d_{m1}, from the proposed uniform distribution range. The random design value of the hydraulic retention time (θ_{m1}) is determined from equation 4.12. The computer program uses the empirical equations of Von Sperling (1999) to calculate the random design value of the effluent faecal coliform concentration in the first maturation pond. The equations, 4.19, 4.20, 4.21, 4.22 and 4.23, are as follows:

$$d_{m1} = \frac{1}{\left(\frac{L}{W}\right)_{m1}}$$

$$K_{FC_{20}\,m1} = 0.917 H_{m1}^{-0.877} \theta_{m1}^{-0.329}$$

$$K_{FC_{T\,m1}} = K_{FC_{20m1}} \phi^{(T-20)}$$

$$\left(N_e\right)_{m1} = \left(N_e\right)_f x \left(\frac{4a_{m1} e^{\frac{1}{2d_{m1}}}}{\left(1+a_{m1}\right)^2 e^{\frac{a_{m1}}{2d_{m1}}} - \left(1-a_{m1}\right)^2 e^{\frac{-a_{m1}}{2d_{m1}}}} \right)$$

$$a_{m1} = \sqrt{(1 + 4K_{FC_T m1}\theta_{m1} d_{m1})}$$

where

the subscript "*m1*" refers to first maturation pond

$(N_e)_{m1}$ = effluent faecal coliform concentration (per 100 ml)

$(N_e)_f$ = influent faecal coliform concentration (per 100 ml)

$K_{FC_{T\,m1}}$ = faecal coliform die-off rate at temperature T °C

$K_{FC_{20}\,m1}$ = faecal coliform die-off rate at 20°C

H_{m1} = pond depth (m)

ϕ = temperature coefficient of faecal coliform removal = 1.07

d_{m1} = dispersion numbers

L = pond length (m)

W = pond breath (m)

θ_{m1} = hydraulic retention time (days)

T = air pond temperature (°C)

For the next run of the simulation, the computer program selects new sets of the random design values of temperature T, influent faecal coliform concentration $(N_e)_f$, temperature coefficient of faecal coliform removal ϕ, dispersion numbers d_{m1}, and hydraulic retention time θ_{m1}, from the proposed uniform distribution range.

5.3.2.8 Random design computation of the effluent faecal coliform concentration in second and subsequent maturation ponds using the empirical equations of Von Sperling (1999)

The computer program uses Monte Carlo simulations to compute the random design value of the effluent faecal coliform concentration in the second and subsequent maturation ponds. The program selects randomly the input design variables of temperature *T*, influent faecal coliform concentration $(N_e)_{m1,}$ temperature coefficient of faecal coliform removal ϕ, and dispersion numbers d_m, from the proposed uniform distribution range. The random design value of the hydraulic retention time (θ_m) is selected randomly from the minimum design range of 3 - 5 days. The computer program designs the number of second and subsequent maturation ponds by a process of iteration. The program determines the random design value of the effluent faecal coliform concentration in second and subsequent maturation ponds by using the empirical equations of Von Sperling (1999), as presented in equation 4.24, 4.25, 4.26, 4.27 and 4.28 as follows:

$$d_m = \frac{1}{\left(\frac{L}{W}\right)_m}$$

$$K_{FC_{20\,m}} = 0.917 H_m^{-0.877} \theta_m^{-0.329}$$

$$K_{FC_{T\,m}} = K_{FC_{20m}} \phi^{(T-20)}$$

$$(N_e)_m = (N_e)_{m1} x \left(\frac{4a_m e^{\frac{1}{2d_m}}}{(1+a_m)^2 e^{\frac{a_m}{2d_m}} - (1-a_m)^2 e^{\frac{-a_m}{2d_m}}} \right)^n$$

$$a_m = \sqrt{(1 + 4K_{FC_T m} \theta_m d_m)}$$

where:

the subscript "*m*" refers to second and subsequent maturation ponds

$(N_e)_m$ = effluent faecal coliform concentration (per 100 ml)

$(N_e)_{m1}$ = influent faecal coliform concentration (per 100 ml)

$K_{FC_{T\,m}}$ = faecal coliform die-off rate at temperature $T\,^{o}$C

$K_{FC_{20\,m}}$ = faecal coliform die-off rate at 20^{o}C

H_m = pond depth (m)

ϕ = temperature coefficient of faecal coliform removal = 1.07

d_m = dispersion numbers

L = pond length (m)

W = pond breath (m)

θ_m = hydraulic retention time = 3–5 days

T = air pond temperature (^oC)

n = number of second and subsequent maturation ponds

During the next run of the simulation, the program selects new sets of the random design values of temperature T, influent faecal coliform concentration $(N_e)_{m1}$, temperature coefficient of faecal coliform removal ϕ, dispersion numbers d_m, and hydraulic retention time θ_m, from their proposed uniform distribution range. These random design values are used to compute the next random design value of effluent faecal coliform concentration.

5.3.2.8.1 Design of the number of second and subsequent maturation ponds based on the 95-%ile of effluent faecal coliform concentration (<1000FC per 100ml)

The computer program selects "one" second maturation pond at a time when carrying out the Monte Carlo simulations. The design calculations are repeated with this "one" second maturation pond, using new sets of input design parameters selected from the proposed uniform distribution range. The design calculations are repeated until the final (1000th) simulation has been completed.

The computer program is designed to prepare the frequency cumulative data of the random design values of the effluent faecal coliform concentration obtained from the total number of simulations (1000). The 95-%ile value of effluent faecal coliform is selected from the frequency cumulative data and is compared with the standard faecal coliform concentration of 1000 FC per 100ml for unrestricted crop irrigation. If the selected 95-%ile value of the effluent faecal coliform concentration is more than 1000 FC per100ml, the computer program adds one maturation pond to the second maturation pond. The design calculations are then repeated with these "two" second maturation ponds until the number of required simulations (1000) are run. The computer program once again compares the selected 95-%ile value of effluent faecal coliform concentration with the standard effluent faecal coliform concentration of 1000 FC per 100ml. If the selected 95-%ile value of FC is less than 1000 FC per 100ml, the computer program provides the design solution for the number of second and subsequent maturation ponds.

5.3.2.9 Random design computation of the anaerobic pond area, facultative pond area, first maturation pond area, second and subsequent maturation ponds areas (Mara et al., 1992)

The design of WSP area is completed when the 95-%ile value of the effluent faecal coliform concentration in the last maturation pond is less than 1000 FC per 100ml. The computer program uses Monte Carlo simulations to compute the random design area of the anaerobic pond, facultative pond, first maturation pond and the second and subsequent maturation ponds. The computer program selects randomly the design values of flow rate Q_i and net evaporation rate e from their proposed uniform distribution range. In addition, the computer program uses the computed random values of the hydraulic retention times in anaerobic, facultative and maturation ponds when designing WSP area. The depths of the WSP are treated as constant values. The program determines the random design area of the anaerobic pond as shown in equation 5.15:

$$A_a = \frac{\theta_a Q_i}{H_a} \tag{5.15}$$

where:
the subscript "a" refers to anaerobic pond
The variables A_a, θ_a, Q_i are random design values
A_a = area of anaerobic pond (m^2)
θ_a = hydraulic retention time of anaerobic pond (days)
Q_i = influent flow (m^3/day)
H_a = pond depth of anaerobic pond (m)

The computer program uses Monte Carlo simulations to compute the random design area of the facultative pond as shown in equation 5.16:

$$A_f = \frac{2Q_i\theta_f}{\left(2H_f + 0.001e\theta_f\right)} \tag{5.16}$$

where
the subscript "f" refers to facultative pond

the variables A_f, θ_f, Q_i and e are random design values

A_f = area of facultative pond (m^2)

θ_f = hydraulic retention time of facultative pond (days)

Q_i = influent flow (m^3/day)

H_f = pond depth of facultative pond (m)

e = net evaporation rate (mm/day)

Monte Carlo simulation is used by the program to compute the random design area of the first maturation pond as shown in equations 5.17 and 5.18:

$$(Q_e)_f = Q_i - 0.001eA_f \tag{5.17}$$

$$A_{m1} = \frac{2(Q_e)_f \theta_{m1}}{(2H_{m1} + 0.001e\theta_{m1})} \tag{5.18}$$

where:

the subscript "$m1, f$" refers to first maturation pond and facultative pond

the variables A_{m1}, θ_{m1}, $(Q_e)_f$ and e are random design values

A_{m1} = area of first maturation pond (m^2)

θ_{m1} = hydraulic retention time of first maturation pond (days)

$(Q_e)_f$ = influent flow rate into first maturation pond (m^3/day)

H_{m1} = pond depth of first maturation pond (m)

e = net evaporation rate (mm/day)

The program uses Monte Carlo simulations to compute the random design area of the second and subsequent maturation ponds as shown in equations 5.19 and 5.20:

$$(Q_e)_{m1} = (Q_e)_f - 0.001eA_{m1} \tag{5.19}$$

$$A_m = \frac{2(Q_e)_{m1} \theta_m}{(2H_m + 0.001e\theta_m)} \tag{5.20}$$

where

the subscript "m, $m1, f$" refers to second and subsequent maturation pond, first maturation pond and facultative pond, respectively;

the variables A_m, θ_m, $(Q_e)_{m1}$ and e are random design values;

A_m = area of second and subsequent maturation ponds (m^2)

θ_m = hydraulic retention time of second and subsequent maturation ponds (days)

$(Q_e)_{m1}$ = influent flow rate into second maturation pond (m^3/day)

H_m = pond depth of second and subsequent maturation ponds (m)

e = net evaporation rate (mm/day)

5.4 Analysis of the output design data

The output design data of WSP calculated by Monte Carlo simulations are presented in statistical form. The computer program presents the designs of WSP area, hydraulic retention time and effluent faecal coliform concentration in 50-%ile and 95-%ile form. In addition, frequency cumulative curves and histograms are generated that show the distribution of the effluent faecal coliform concentration.

The significance of presenting the effluent faecal coliform concentration in percentile form and as a frequency cumulative curve is that the designer is able to choose the acceptable percentile value, which then forms the basis for designing the WSP area. The choice of the percentile value depends on the acceptable health risk exposed to the public if the final treated effluent is to be used for unrestricted crop irrigation. Cost is also a factor in this choice. If the available cost to procure the WSP project is not adequate, a lower percentile value (50-%ile) of the effluent faecal coliform concentration can be chosen to design the area of WSP. The resulting area of WSP will be lower than the area of WSP designed on the basis of a higher percentile value (95-%ile).

An example is presented in this section to illustrate modern methods for designing WSP, with special emphasis on the presentation of the output design data. The example is the design of a system to treat domestic wastewater that has typical characteristics of that found in developing countries, and the treated wastewater is to be used for unrestricted crop irrigation. The average deterministic single values of the input design parameters are allowed to vary by ± 20% to establish the range of their variation, while some input design parameters such as pond depth are kept constant. The range of the input design parameters is presented as follows:

Per capita BOD contribution (g/person day) = (32, 48)

Per capita wastewater production (*l*/person day) = (96, 144)

Design population = (80000, 120000)

Net evaporation (mm/day) = (3.2, 4.8)

Depth of anaerobic pond (m) = (4.0, 4.0)

Depth of facultative pond (m) = (1.5, 1.5)

Depth of first maturation pond (m) = (1.0, 1.0)

Depth of second and subsequent maturation ponds (m) = (1.0, 1.0)

Number of influent faecal coliform concentration (per 100 ml) = (7×10^7, 1.2×10^8)

Temperature (^oC) = (16, 24)

Temperature coefficient of faecal coliform removal = (0.856, 1.284)

First-order faecal coliform removal constant rate in anaerobic pond (day^{-1}) = (1.6, 2.4)

Dispersion numbers in facultative pond $\dfrac{1}{\left(\dfrac{L}{B}\right)_f}$ = (0.08, 0.12)

Dispersion numbers in first maturation pond $\dfrac{1}{\left(\dfrac{L}{B}\right)_{m1}}$ = (0.04, 0.06)

Dispersion numbers in 2^{nd} and subsequent maturation ponds $\dfrac{1}{\left(\dfrac{L}{B}\right)_m}$ = (0.04, 0.06)

Minimum hydraulic retention time in 2^{nd} and subsequent maturation ponds (days) = (3, 5)

Minimum hydraulic retention time anaerobic pond (days) = (1)

Minimum hydraulic retention time in facultative pond (days) = (4)

Minimum hydraulic retention time in maturation pond (days) = (3)

Number of simulations = 1000

The Visual Basic program was run with 1000 simulations using the range of the input design parameters shown above. The results of the output design data are presented in Tables 5.1, 5.2 and 5.3.

Table 5.1 Effluent faecal coliform concentration from Monte Carlo simulations (1000 runs)

Statistical parameter	Number of faecal coliforms per 100 ml of effluent
Mean	260
Minimum	0
Maximum	51,000
50%-ile value	2
95%-ile value	780

Table 5.2 Area of WSP from Monte Carlo simulations (1000 runs) (m^2)

Statistical parameter	Anaerobic	Facultative	1^{st} Maturation	Subsequent maturation ponds; no. of maturation ponds = 6
Mean	3,647.20	68,493.83	55,865.74	47,502.15
Minimum	1,944.09	25,470.52	22,611.80	22,462.24
Maximum	6,326.91	145,044.06	117,279.76	80,283.23
50%ile value	3,553.48	63,807.82	49,635.81	46,365.51
95-%ile value	5,345.29	118,253.87	98,280.69	69,407.90

Table 5.3 Hydraulic retention time of WSP from Monte Carlo simulations (1000 runs) (days)

Statistical parameter	Anaerobic	Facultative	1^{st} Maturation	Subsequent maturation ponds; no. of maturation ponds = 6
Mean	1.16	8.28	4.60	3.98
Minimum	1.00	4.87	3.00	3.00
Maximum	1.51	13.39	7.30	5.00
50-%ile value	1.10	7.81	3.47	4.00
95-%ile value	1.45	12.56	7.01	4.88

The design output of the effluent faecal coliform concentration shown in Table 5.1 demonstrates that a series of anaerobic pond, facultative pond and seven maturation ponds is capable of producing effluent which would satisfy the criteria for unrestricted crop irrigation based on a 95-%ile (95-%ile FC = 782 per 100 ml – ie, less than 1000 per 100 ml). The model predicts that 95% of the 1000 calculated values of the effluent faecal coliform concentration would be less than 1000 FC per 100ml, assuming that the average values of the input design parameters vary by ±20%.

The design output of WSP area and hydraulic retention time are presented in Tables 5.2 and 5.3 respectively. Based on 95-%ile, the area of the WSP can be designed by using the design values indicated by the 95-%ile row. If designed in this way it can be concluded that the WSP would have a 95% probability that the effluent faecal coliform concentration would be less than 1000 FC per 100ml.

Figure 5.2 shows the design output of the effluent faecal coliform concentration presented as a frequency cumulative curve and histogram. The histogram shows that a range of 20–60 FC per 100 ml is the highest predicted frequency (818) of the effluent faecal coliform concentration from the WSP. The cumulative frequency curve confirms the 95%ile of the effluent faecal coliform concentration provided by the computer program (Table 5.1) as 782 FC per 100ml. The designer can choose any required percentile value from the graph to design the area of WSP depending on the available cost and the acceptable health risk.

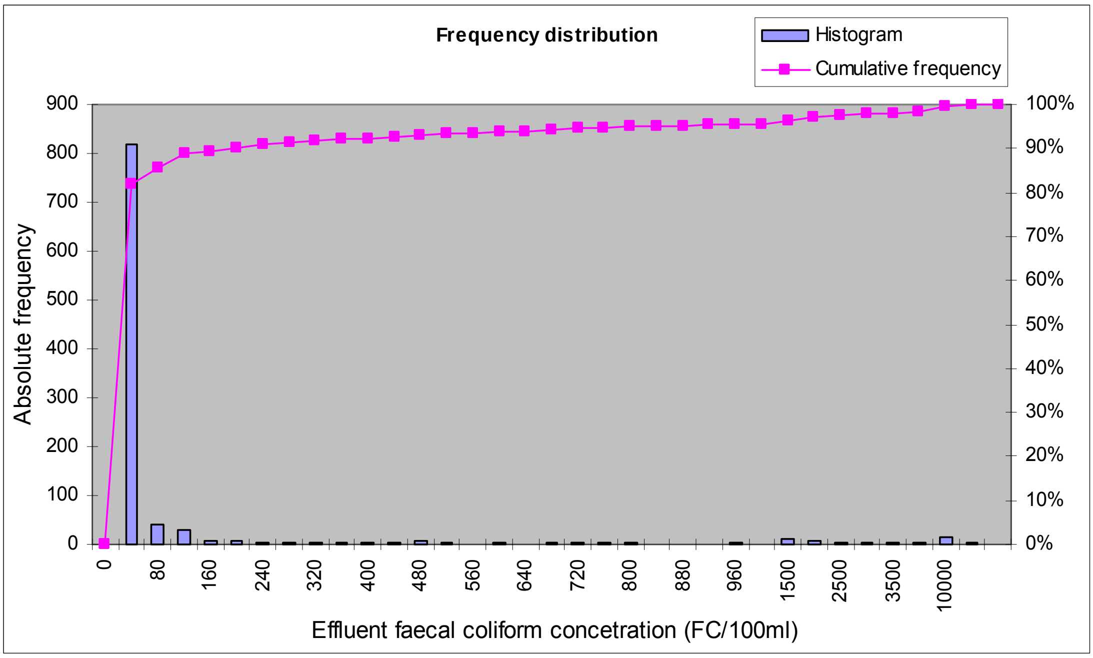

Figure 5.2 Frequency cumulative curve and histogram of the effluent FC from Monte Carlo simulations (1000 runs)

In summary, this chapter has shown how the a Visual Basic computer program has been developed to design WSP based on modern design methods. The computer program has incorporated Monte Carlo simulations, an inverse cumulative function of the uniform probability distribution range of input design parameters and a random number function. The chapter has demonstrated how Monte Carlo simulations are used to design anaerobic ponds, facultative ponds and maturation ponds based on a proposed range of the input design parameters. The chapter has presented a rational approach for designing maturation ponds based on a 95%ile of the effluent faecal coliform concentration. In conclusion, the computer program that has been developed is effective and reliable tool for designing WSP because of its ability to provide output design data in a statistical format. Furthermore, the computer program is user-friendly and is simple to run due to its compatibility with Microsoft Excel Spreadsheet. The program presents a realistic approach to designing maturation ponds based on 95%ile of effluent faecal coliform concentration rather than relying on a figure for mean effluent faecal coliform concentration, which is the case for traditional design methods. The proposed method (95-%ile of FC) of designing maturation ponds presents an accurate way of determining the risk to public health if the final effluent is to be used for unrestricted irrigation of crops. The computer program provides a statistical output design data, which helps the designer to make a decision regarding the size of WSP. The designer can select the size of the WSP system based on the available funds and the acceptable health risk to the public.

CHAPTER 6

THE EFFICIENCY OF THE MODERN DESIGN METHOD OF WASTE STABILIZATION PONDS

Chapter 5 has shown that modern methods represent a realistic approach to designing WSP, given that they enable decisions to be taken that are based upon the available funds for the project and an assessment of the acceptable risks to public health if the final effluent is to be used for unrestricted crop irrigation. The modern design method uses range values for the input design parameters – average single values of the input design parameters are allowed to vary by a suitable margin that reflects levels of uncertainty.

Von Sperling (1996, 2002) proposed that a sensitivity analysis should be carried out during the design stage of WSP to identify those input parameters that significantly influence the design output. Such an analysis enables resources to be targeted at the most important design parameters, aiding the economic design of the system.

In this chapter, the efficiency of the modern design method of WSP is tested by the application of the sensitivity analysis on the design output. The modern method is also compared with the traditional design method in terms of the flexibility of the approaches. The land requirement for constructing the WSP, the cost of procuring the WSP and safety factors are evaluated.

6.1 Sensitivity analysis

Von Sperling (1996, 2002) carried out sensitivity analysis on the design output of WSP in order to identify the input design parameters that significantly influence the design output. He suggested that sensitivity analysis acts as a framework for decision making during the design process. If the designer knows which input parameters significantly influence the design output, the designer can confidently argue for adequate resources to be made available for accurately assessing these input parameters. The modern design method is a flexible design approach and it allows sensitivity analysis to be applied in this way. Sensitivity analysis is especially important when WSP are designed to treat wastewater that will eventually be used for the unrestricted irrigation of crops. The analysis indicates the sensitivity of the effluent faecal coliform concentration to variations of the input design parameters. Importantly, sensitivity analysis also identifies those input parameters that have no influence on the design output. This prevents needless expenditure on factors that do not affect the final output. The procedure for sensitivity analysis as proposed by Von Sperling (1996) is presented in Figure 6.1.

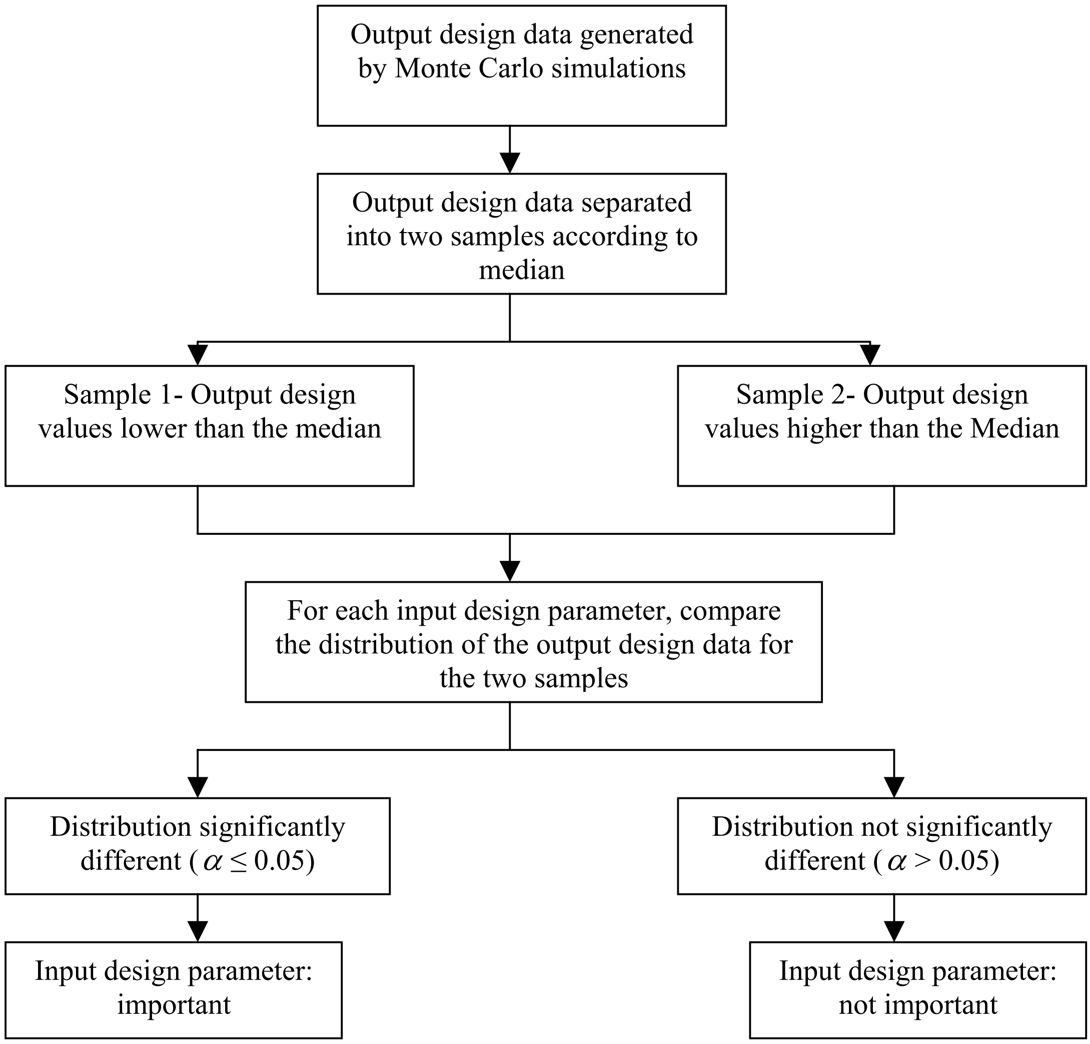

Figure 6.1 Procedure for the sensitivity analysis

Source: Von Sperling (1996)

The procedures for the sensitivity analysis is carried out by using Monte Carlo simulations to generate the design output, whereby one input design parameter is allowed to vary within its proposed range while the rest of the input design parameters are assigned single average values. The sensitivity analysis on the effluent faecal coliform concentration and area of WSP is effected by varying each input design parameter by ±20% from its average design value while the remaining input design parameters are assigned constant values. The Visual Basic computer program was run with "1000" simulations in order to provide the design output data of effluent faecal coliform concentration and area of WSP. These two sets of output design data are separated into two samples according to their median. One sample of the output design data has "500" values lower than the median; while the other sample has "500" values higher than the median.

The distributions of the sample values from the median are determined by taking the mathematical absolute values of the differences between the sample values and the median. Since the resulting distributions are not known, the two-sample test is based on non-parametric statistical method due to its characteristic of being distribution free (Massey, 1951; Marascuilo and McSweeney, 1977).

The Kolmogorov-Smirnov test is used to carry out the sensitivity analysis on the output design data of the two distribution samples. This test is the non-parametric statistical method and calculates the maximum separation between the cumulative distribution functions of the two samples. The Kolmogorov-Smirnov test is very sensitive to any differences in the distribution functions. The test is based on the null hypothesis (H_O) that two distribution samples are the same if the probability of being the same is higher than the proposed significance level (α). Under these conditions, the input design parameter does not significantly influence the separation of the output design data from their median. If the probability of being the same is less than the assigned significance level value (α), the input design parameter is considered to significantly influence the output design data

The Kolmogorov-Smirnov two-sample test on the two distribution samples of the output design data was carried out by using a statistical software program (SPSS 11.5 for Windows). The critical difference of the cumulative functions of the two distribution samples is determined by the equation developed by Massey (1951) at significance level of 5% as follows:

$$F(x) - S(x) = 1.36\sqrt{\frac{N_1 + N_2}{N_1 N_2}} \qquad (6.1)$$

where

$F(x)$ = distribution function for sample 1

$S(x) =$ distribution function for sample 2

N_1 = number of observations in distribution sample 1 = 500

N_2 = number of observations in distribution sample 2 = 500

Table 6.1 shows the results of the Kolmogorov-Smirnov two-sample test obtained from the statistical software program SSPS (release 11.5 for Windows) on the effluent faecal coliform concentration when each input design parameter varies by ±20% from its average design value while the rest of the input design parameters have constant single values.

Table 6.1 Results from the Kolmogorov-Smirnov two-sample test for the sensitivity analysis on the effluent faecal coliform concentration

Input design parameter	Observed difference $F(x) - S(x)$	Critical difference $1.36\sqrt{\frac{N_1 + N_2}{N_1 N_2}}$	Two-sided probability (α) of the samples being the same α = 0.05	Significance of the input design parameter on the design output
Per capita BOD	0.408	0.086	0.000	**critical**
Per capita wastewater	0.402	0.086	0.000	**critical**
Population	0.000	0.086	1.000	unimportant
Evaporation	0.154	0.086	0.000	**critical**
Influent FC concentration	0.082	0.086	0.069	unimportant
Temperature	0.470	0.086	0.000	**critical**
FC removal constant rate in anaerobic pond	0.204	0.086	0.000	**critical**
Temperature coefficient	0.000	0.086	1.000	unimportant
Dispersion numbers in facultative pond	0.118	0.086	0.002	**critical**
Dispersion numbers in first maturation	0.072	0.086	0.150	unimportant
Dispersions numbers in second “n” maturation	0.070	0.086	0.172	unimportant

The results of Table 6.1 show that effluent faecal coliform concentration is significantly influenced by per capita BOD, per capita wastewater flow, temperature, net evaporation rate, faecal coliform removal constant rate in anaerobic pond and the dispersion numbers in facultative ponds at the assumed proposed range of the input design parameters (average value ±20%).

The input design parameters of per capita BOD, per capita wastewater, and net evaporation rate are related to the random computation of the hydraulic retention time in anaerobic, facultative and first maturation ponds (Chapter 5). It has been shown that the empirical equations of Mara (2002) and Von Sperling (1999) use hydraulic retention time in modelling faecal coliform removal in WSP. Variations in these input design parameters significantly influence the effluent faecal coliform concentration.

The faecal coliform removal constant rate in anaerobic ponds and temperature are also critical input design parameters that significantly influence the effluent faecal coliform concentration. These input design parameters are used in the empirical equations of Mara (2002) and Von Sperling (1999) for modelling faecal coliform removal in WSP. Variations of these critical input design parameters cause the effluent faecal coliform concentrations to divert from their expected values. The designer should therefore ensure that adequate resources are allocated to the accurate determiniation of these important input design parameters.

Von Sperling (2002) carried out sensitivity analysis on the effluent faecal coliform concentration in WSP by using the non-parametric Mann-Whitney U test. He observed that liquid temperature and per capita wastewater did not significantly influence the effluent faecal coliform concentration, whereas the influent faecal coliform concentration was the critical input design parameter that significantly influenced the effluent faecal coliform concentration in maturation ponds. He used the hydraulic retention times and the faecal coliform removal constant rate as the input design parameters.

The results described in the present report are at variance with those of Von Sperling (2002), with a similar range of input design parameters. One reason for this discrepancy could by that the non-parametric tests used for the sensitivity analyses were different (Kolmogorov-Smirnov and Mann-Whitney U test).

However, Von Sperling's sensitivity analysis is based on a facultative pond and three maturation ponds designed by adopting the typical design procedures for WSP, rather than a design based on first principles. For instance, the facultative pond and the three maturation ponds were assumed to have typical hydraulic retention times of 20 days and 4 days respectively. The number of maturation ponds was assumed to be "3". Furthermore, the faecal coliform removal constant rate in the facultative and maturation ponds was assumed to be an input design parameter. These suggested variables are not input design parameters, as proposed by Von Sperling, but are output design parameters that depend upon each other and upon other input design parameters. Von Sperling did not include an anaerobic pond to model faecal coliform removal in the typical WSP. It is possible that these factors could have influenced the results of the sensitivity analysis carried out by Von Sperling. In the present study, the results of the sensitivity analysis on the effluent faecal coliform concentration could be more realistic given that the analysis has been based on first principles for the modern design of WSP. The sensitivity analysis is based on a properly designed anaerobic pond, a facultative pond and five maturation ponds. The resulting 95-%ile value of the effluent faecal coliform concentration in all the sensitivity analysis procedures was less than 1000 FC per 100ml.

Table 6.2 shows the results of the Kolmogorov-Smirnov two-sample test obtained from the

Table 6.2 Results from the Kolmogorov-Smirnov two-sample test for the sensitivity analysis on the area of the anaerobic pond

Input design parameter	Observed difference $F(x)-S(x)$	Critical difference $1.36\sqrt{\frac{N_1+N_2}{N_1N_2}}$	Two-sided probability (α) of the samples being the same α = 0.05	Significance of the input design parameter on design output
Per capita BOD	0.478	0.086	0.000	**critical**
Per capita wastewater	0.156	0.086	0.000	**critical**
Population	0.094	0.086	0.024	**critical**
Evaporation	0.000	0.086	1.000	unimportant
Influent FC concentration	0.000	0.086	1.000	unimportant
Temperature	0.666	0.086	0.000	**critical**
FC removal constant rate in anaerobic pond	0.000	0.086	1.000	unimportant
Temperature coefficient	0.000	0.086	1.000	unimportant
Dispersion numbers in facultative pond	0.000	0.086	1.000	unimportant
Dispersion numbers in first maturation	0.000	0.086	1.000	unimportant
Dispersions numbers in second “n” maturation	0.070	0.0860	0.172	unimportant

statistical software program SSPS (11.5 for Windows) for the area of anaerobic pond when each input design parameter varies by ±20% from its average design value while the rest of the input design parameters have constant single values. Table 6.2 shows that the area of an anaerobic pond is significantly influenced by the per capita BOD, per capita wastewater flow, temperature and population when these critical input design parameters vary by ±20% from their average design values.

Table 6.3 shows the results of the Kolmogorov-Smirnov two-sample test obtained from the statistical software program SSPS (11.5 for Windows) for the area of a facultative pond when each of the input design parameters varies by ±20% from its average design value, while the rest of the input design parameters have constant single values.

Table 6.3 Results from the Kolmogorov-Smirnov two-sample test for the sensitivity analysis on the area of the facultative pond

Input design parameter	Observed difference $F(x) - S(x)$	Critical difference $1.36\sqrt{\frac{N_1 + N_2}{N_1 N_2}}$	Two-sided probability (α) of the samples being the same $\alpha = 0.05$	Significance of the input design parameter on design output
Per capita BOD	0.074	0.086	0.129	unimportant
Per capita wastewater	0.164	0.086	0.000	**critical**
Population	0.096	0.086	0.020	**critical**
Evaporation	0.000	0.086	1.000	unimportant
Influent FC concentration	0.000	0.086	1.000	unimportant
Temperature	0.372	0.086	0.000	**critical**
FC removal constant rate in anaerobic pond	0.000	0.086	1.000	unimportant
Temperature coefficient	0.000	0.086	1.000	unimportant
Dispersion numbers in facultative pond	0.000	0.086	1.000	unimportant
Dispersion numbers in first maturation	0.000	0.086	1.000	unimportant
Dispersions numbers in second “n” maturation	0.070	0.0860	0.172	unimportant

Table 6.3 shows that the area of a facultative pond is significantly influenced by the per capita wastewater flow, temperature and population when these input design parameters vary by ±20% from their average design values. These input design parameters are related to the design equation that is employed in designing the facultative pond.

Table 6.4 shows the results of the Kolmogorov-Smirnov two-sample test obtained from the statistical software program (SSPS 11.5 for Windows) for the area of the first maturation pond when each of the input design parameters varies by ±20% from its average design value while the rest of the input design parameters have constant single values .

Table 6.4 Results from the Kolmogorov-Smirnov two-sample test for the sensitivity analysis on the area of the first maturation pond

Input design parameter	Observed difference $F(x) - S(x)$	Critical difference $1.36\sqrt{\frac{N_1 + N_2}{N_1 N_2}}$	Two-sided probability (α) of the samples being the same α = 0.05	Significance of the input design parameter on design output
Per capita BOD	0.076	0.086	0.111	unimportant
Per capita wastewater	0.164	0.086	0.000	**critical**
Population	0.096	0.086	0.020	**critical**
Evaporation	0.030	0.086	0.978	unimportant
Influent FC concentration	0.000	0.086	1.000	unimportant
Temperature	0.968	0.086	0.000	**critical**
FC removal constant rate in anaerobic pond	0.000	0.086	1.000	unimportant
Temperature coefficient	0.000	0.086	1.000	unimportant
Dispersion numbers in facultative pond	0.000	0.086	1.000	unimportant
Dispersion numbers in first maturation	0.000	0.086	1.000	unimportant
Dispersions numbers in second "n" maturation	0.000	0.086	1.000	unimportant

Table 6.4 shows that the area of the first maturation pond is significantly influenced by the per capita wastewater flow, temperature and population when these input design parameters

vary by ±20% from their average design values. These input design parameters are related to the design equation that is employed in designing the first maturation ponds.

Table 6.5 shows the results of the Kolmogorov-Smirnov two-sample test obtained from the statistical software program SSPS (11.5 for Windows) for the area of second and subsequent maturation ponds when each of the input design parameters varies by ±20% from its average design value while the rest of the input design parameters have constant single values.

Table 6.5 Results from the Kolmogorov-Smirnov two-sample test for the sensitivity analysis on the area of the second maturation pond

Input design parameter	Observed difference $F(x)-S(x)$	Critical difference $1.36\sqrt{\frac{N_1+N_2}{N_1N_2}}$	Two-sided probability (α) of the samples being the same α = 0.05	Significance of the input design parameter on design output
Per capita BOD	0.076	0.086	0.111	unimportant
Per capita wastewater	0.060	0.086	0.329	unimportant
Population	0.096	0.086	0.020	**critical**
Evaporation	0.032	0.086	0.960	unimportant
Influent FC concentration	0.000	0.086	1.000	unimportant
Temperature	0.820	0.086	0.000	**critical**
FC removal constant rate in anaerobic pond	0.000	0.086	1.000	unimportant
Temperature coefficient	0.000	0.086	1.000	unimportant
Dispersion numbers in facultative pond	0.000	0.086	1.000	unimportant
Dispersion numbers in first maturation	0.000	0.086	1.000	unimportant
Dispersions numbers in second "n" maturation	0.000	0.086	1.000	unimportant

Table 6.5 shows that the area of the second maturation pond is significantly influenced by the population and temperature when these input design parameters vary by ±20% from their average design values.

The results of sensitivity analysis on the area of WSP show that the area of anaerobic ponds, facultative ponds and maturation ponds are significantly influenced by the per capita wastewater flow, population and temperature. Only anaerobic ponds are significantly influenced by the per capita BOD. These results of the sensitivity analysis help the design engineer to allocate adequate resources for the accurate measurement of these critical input design parameters in order to design a WSP system economically.

The results of the sensitivity analysis on the area of WSP agree with some of the critical input design parameters found by Von Sperling (1996). He observed that population and the per capita wastewater flow are the critical input design parameters that significantly influence the area of facultative ponds. He observed that temperature is not one of the critical input design parameters that significantly influences the area of the facultative pond.

The author of this present report has identified temperature as one of the critical input design parameters that significantly influences the area of anaerobic, facultative and maturation ponds. Von Sperling (1996) based his sensitivity analysis procedures on the area of the facultative pond, which was designed by using a constant value for surface BOD loading of 200kg BOD/ha/day. This could be one of the weaknesses of the sensitivity analysis adopted by Von Sperling (1996). The author of the present report has applied the sensitivity analysis procedures on a putative facultative pond that has been designed by using the random design value of the surface BOD loading to mimic the variation of the temperature. It is suggested that this could be a more rational approach than that of Von Sperling (1996). Temperature, therefore, should be treated as one of the critical input design parameters that significantly influences the area of the WSP.

6.2 Comparison of WSP designed by traditional methods with modern methods in terms of the faecal coliform removal efficiency

The two methods of designing WSP, traditional and modern, are compared by designing a model WSP system with similar input parameters whose final effluent satisfies the criteria for unrestricted irrigation of crops. The influent wastewater has characteristics typical of that found in developing countries. The traditional design method is based on the procedures of Marais (1974) procedures, while the modern method employs Monte Carlo simulations and the empirical equations of Mara (2002) and Von Sperling (1999) based on a dispersed hydraulic flow regime. The modern design method treats the input design parameters as range values.

6.2.1 The design output of the traditional design method

The following input design parameters have been selected to design a WSP system based on the traditional design method for producing effluent suitable for unrestricted crop irrigation:

Per capita BOD contribution = 40 g/person day
Per capita wastewater production = 120 *l*/person day
Design population = 100,000
Net evaporation = 6 mm/day
Depth of anaerobic pond = 4 m
Depth of facultative pond = 1.5 m
Depth of first maturation pond = 1.0 m
Depth of second and subsequent maturation ponds = 1.0 m
Influent faecal coliform concentration = 1×10^8 per 100 ml
Temperature = 18°C
Temperature coefficient for faecal coliform removal = 1.19
First-order faecal coliform removal constant rate in WSP = 2.6 day^{-1}

Design of the anaerobic pond

From Table 3.1, the volumetric organic loading rate is given by:
$\lambda_v = 20T\text{-}100 = (20 \text{ x } 18) - 100 = 260 \text{ g/m}^3 \text{ day}$

The anaerobic pond volume:

Influent BOD, $L_i = \dfrac{40x1000}{120} = 333.33 mg/l$

Design wastewater flow, $Q = \dfrac{100{,}000x120}{1000} = 12{,}000 m^3/day$

The pond volume = $\dfrac{L_i Q}{\lambda_V} = \dfrac{333.33x12{,}000}{260} = 15{,}385 m^3$

The anaerobic pond area = $\dfrac{15{,}385}{4} = 3{,}846.25 m^2$

The hydraulic retention time = $\dfrac{L_i}{\lambda_V} = \dfrac{333.33}{260} = 1.28 days$

The BOD removal in anaerobic pond is given by Table 3.2

R = $2T + 20 = 56\%$

Design of the facultative pond

The design surface loading is given by equation 3.23, as follows:

$$\lambda_{sf} = 350(1.107 - 0.002T)^{T-25}$$

$$\lambda_{sf} = 350(1.107 - 0.002x18)^{18-25} = 216kg/ha/day$$

The facultative pond area is calculated from equation 3.24, as follows:

$$A_f = \frac{10(0.44x333.33)x12{,}000}{216} = 81{,}480.67m^2$$

The retention time of the facultative pond is calculated from equation 4.11, as follows:

$$\theta_f = \frac{2AH_f}{(2Q - 0.001A_f e)}$$

$$\theta_f = \frac{2x81{,}480.67x1.5}{(2x12{,}000 - 0.001x81{,}480.67x6)} = 10.40\,days$$

effluent flow $=12{,}000 - (0.001x6x81{,}480.67) = 11{,}511.12m^3$

Design of the first maturation pond

The hydraulic retention time is given by equation 4.12, as follows:

$$\theta_{m1} = \frac{10L_i H_{m1}}{0.75\lambda_{sf}}$$

$$\theta_{m1} = \frac{10(0.3x333.33)x1.0}{0.75x216} = 6.17days$$

The area of the first maturation pond is given by equation 5.18, as follows:

$$A_{m1} = \frac{2(Q_e)_f\,\theta_{m1}}{(2H_{m1} + 0.001e\theta_{m1})}$$

$$A_{m1} = \frac{2x11{,}511.12x6.17}{(2x1.0 + 0.001x6x6.17)} = 69{,}732.86m^2$$

Design of the second and subsequent maturation ponds

The value of K_{FC_T} is given by equation 3.28. as follows:

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

$$K_{FC_T} = 2.6(1.19)^{-2} = 1.84 day^{-1}$$

The hydraulic retention time in the second and subsequent maturation ponds is given by equation 3.30 as follows:

$$\theta_m = \frac{\left[\dfrac{N_i}{N_e\left(1+K_{FC_T}\theta_a\right)\left(1+K_{FC_T}\theta_f\right)\left(1+K_{FC_T}\theta_{m1}^{\min}\right)}\right]^{\frac{1}{n}} - 1}{K_{FC_T}}$$

$$\theta_m = \frac{\left[\dfrac{1x10^8}{10^3\left(1+1.84x1.28\right)\left(1+1.84x10.40\right)\left(1+1.84x6.17\right)}\right]^{\frac{1}{n}} - 1}{1.84}$$

combination 1: for n = 1, $\theta_m = 64.5$days

combination 2: for n = 2, $\theta_m = 5.41$days

combination 3: for n = 3, $\theta_m = 2.13$days

The hydraulic retention time for the third combination is less than the minimum retention time of 3 days. Therefore, three maturation ponds, each with a hydraulic retention time of 3 days, have been selected for the design.

The effluent flow from first maturation pond:

Q = $(11{,}511.12 - 0.001x6x69{,}732.86)$ m^3/day = 11,092.80m^3/day

The area of each of the three subsequent maturation ponds is given by equation 5.20, as follows:

$$A_m = \frac{2(Q_e)_{m1}\theta_m}{\left(2H_m + 0.001e\theta_m\right)}$$

$$A_m = \frac{2x11{,}092.80x3.00}{\left(2x1.0 + 0.001x6x3.00\right)} = 32{,}981.57m^2$$

6.2.2 The design output of the modified traditional design method

Marais's faecal coliform removal model for designing maturation ponds has been suggested as being unrealistic (Chapters 3, 4 and 5). In order to compare the efficiency of the traditional design methods with modern design methods, the empirical equation of Mara (2002) for faecal coliform removal in anaerobic ponds is used instead of the equation of Marais (1974), as this is considered more realistic. The design variables of the equation of Mara (2002) are treated as deterministic single values to follow the procedure of traditional design methods. The equation developed by Mara (2002) for faecal coliform removal in anaerobic ponds is given in equation 4.13, as follows:

$$\left(N_e\right)_a = \frac{N_i}{(1 + K_{FC_T}\phi^{(T-20)}\theta_a)}$$

Von Sperling (1999) developed an empirical equation for estimating the faecal coliform removal constant rate (K_{FC_T}) in facultative and maturation ponds. The equation is based on the assumption of complete mixing. This empirical equation predicted accurately the effluent faecal coliform concentration of 33 ponds in Brazil (R^2 = 0.951) for the completely mixed model. This equation replaces that of Marais (1974) for calculating faecal coliform removal in facultative and maturations ponds.

The empirical equation of Von Sperling (1999), based on complete mixing, is presented in equation 6.2, as follows:

$$K_{FC_{20}} = 1.608xH^{-0.877}\theta^{-0.329} + \left[7.656x10^{-4}xH^{-3.674}\theta^{1.811}\left(\frac{L}{B}\right)^{1.509}\right] \tag{6.2}$$

$$K_{FC_T} = K_{FC_{20}}\phi^{T-20} \tag{6.3}$$

where

$K_{FC_{20}}$ = faecal coliform removal constant rate removal at 20°C.

K_{FC_T} = faecal coliform removal constant rate at any temperature

H = pond depth (m)

θ = hydraulic retention time in facultative or maturation ponds (days)

$\frac{L}{B}$ = ratio of pond dimensions = 1 : 10

The temperature coefficient of faecal coliform removal proposed by Marais (1974) is considered relatively high (Chapter 4 and 5). Therefore, a temperature coefficient of 1.07 is used to design the subsequent maturation ponds. Assuming the facultative and maturation ponds have a length-to-width ratio of 1:10, the deterministic single average values of faecal coliform removal in anaerobic, facultative and maturations ponds are as follows:

The K_{FC_T} rate in the anaerobic pond as proposed by Mara (2002):

$$K_{FC_T} = 2.0(1.07)^{18-20} = 1.75 day^{-1}$$

The K_{FC_T} value in the facultative pond as proposed by Von Sperling (1999):

$$K_{FC_{20}} = 1.608x1.5^{-0.877}10.40^{-0.329} + \left(7.656x10^{-4}x1.5^{-3.674}10.40^{1.811}10^{1.509}\right) = 0.908 \text{ day}^{-1}$$

$$K_{FC_T} = 0.908(1.07)^{18-20} = 0.793 day^{-1}$$

The K_{FC_T} value in the first maturation pond as proposed by Von Sperling (1999):

$$K_{FC_{20}} = 1.608x1.0^{-0.877}6.17^{-0.329} + \left(7.656x10^{-4}x1.0^{-3.674}6.17^{1.811}10^{1.509}\right) = 1.551 \text{day}^{-1}$$

$$K_{FC_T} = 1.551(1.07)^{18-20} = 1.355 \text{ day}^{-1}$$

The K_{FC_T} value in subsequent maturation ponds as proposed by Von Sperling (1999) using 3 days minimum hydraulic retention time:

$$K_{FC_{20}} = 1.608x1.0^{-0.877}3.00^{-0.329} + \left(7.65x10^{-4}x1.0^{-3.674}3.00^{1.811}10^{1.509}\right) = 1.301 \text{day}^{-1}$$

$$K_{FC_T} = 1.301(1.07)^{18-20} = 1.136 \text{ day}^{-1}$$

The design of the second and subsequent maturation ponds based on complete mixing, as suggested by Marais (1974), but with a realistic faecal coliform removal constant rate, is presented in equation 6.3 as follows:

$$\frac{N_e}{N_i} = \frac{1}{\left(1 + K_{FC_{Ta}}\theta_a\right)\left(1 + K_{FC_{Tf}}\theta_f\right)\left(1 + K_{FC_{Tm1}}\theta_{m1}\right)\left(1 + K_{FC_{Tm}}\theta_m\right)^n} \tag{6.3}$$

where

The subscripts "*a*, *f*, *m1*, *m*" refer to anaerobic, facultative, first maturation and subsequent maturation ponds

K_{FC_T} = faecal coliform constant removal rate (day^{-1})

θ_a = hydraulic retention time in anaerobic pond (days)

θ_f = hydraulic retention time in facultative pond (days)

θ_{m1} = hydraulic retention time in first maturation pond (days)

θ_m = hydraulic retention time in subsequent maturation pond (days)

N_e = effluent faecal coliform concentration per 100 ml

N_i = influent faecal coliform concentration per 100 ml

$$\frac{N_e}{N_i} = \frac{1}{(1 + 1.75x1.28)(1 + 0.793x10.40)(1 + 1.355x6.17)(1 + 1.136x3)^n}$$

$$\frac{10^3}{1x10^8} = \frac{1}{(3.24)(9.25)(9.36)(4.41)^n}$$

$$\frac{1x10^8}{(3.24)(9.25)(9.36)(4.41)^n} \le 10^3$$

when n = 3, the LHS, effluent faecal coliform concentration = 4,156 FC/100ml

when n = 4, the LHS, effluent faecal coliform concentration = 942 FC/100ml

The design output using the modified traditional design method based on deterministic input design values, complete mixing and realistic values of K_T show that four subsequent maturations ponds should be provided to reduce the faecal coliform concentration to less than 1000 FC per 100 ml (942 FC/100ml). Had the procedure of Marais (1974) been followed, the number of subsequent maturation ponds would have been three. This suggests that such a procedure would result in the design of a system that would be unsuitable for using the final effluent for unrestricted crop irrigation.

6.2.3 The design output of the modern design method

Modern design methods are used to design WSP that satisfy the criteria for unrestricted crop irrigation. The input design parameters are the same as those for the traditional design approach, but are assumed to have a variation of ±20%. In addition, whereas traditional design mehtods take the design temperature as the minimum temperature in the coldest month, the modern method takes a range between the minimum temperature in the coldest month and the maximum in the hottest month.

The Visual Basic program was run with 1000 simulations with the above range of the input design parameters. The output design data are compared with those from traditional design methods in Table 6.6.

Table 6.6 Comparison of the design output of the traditional design method, modified traditional design method and modern design methods of WSP

Design output	Traditional design method	Modified traditional design method	Modern design method based on 50%ile value	Modern design method based on 95%ile value
Mode of input design parameters	Single average input parameters	Single average input parameters	Range of input design parameters	Range of input design parameters
Anaerobic pond	3,846 m^2	3,846 m^2	3,539 m^2	4,966 m^2
Facultative pond	81,481 m^2	81,481 m^2	56,386 m^2	98,942 m^2
1^{st} maturation pond	69,733 m^2	69,733 m^2	44,567 m^2	88,348 m^2
2^{snd} & subsequent maturation ponds	32,982 m^2	32,982 m^2	40,912 m^2	57,660 m^2
No. of subsequent maturation ponds	3	4	6	6
No. of effluent FC concentration/100ml	1000	942	3	447

Table 6.6 shows that the traditional design method of WSP employing the equations of Marais (1974) indicate that three subsequent maturation ponds are required to remove the influent faecal coliform concentration to about 1000 FC per 100 ml. However, the temperature-dependent model of Marais (1974) has been criticized as being unrealistic (Chapters 4 and 5). Therefore, the design of the second and subsequent maturation ponds has been based on a modified traditional design method employing the empirical equations of Von Sperling (1999) and Mara (2002), which assume complete mixing, as proposed by Marais (1974). This modified traditional design method shows that four subsequent maturation ponds are required to remove the influent faecal coliform concentration to less than 1000 FC per 100ml (942 FC/100ml), based on the deterministic values of the input design parameters. This suggests that the traditional design method based on Marais's procedure overestimates the efficiency of removing faecal coliforms, results in an inadequate number of subsequent maturation ponds, and thereby potentially exposes the public to health risks if the treated wastewater were used for unrestricted crop irrigation.

Table 6.6 shows the modern design approach calculates that six subsequent maturation ponds are needed, based on 50-%ile and 95-%ile values, assuming that all input design parameters vary by ±20% from their average design values. The 50-%ile value of the effluent faecal coliform concentration is given as 3 FC per 100 ml, while the 95-%ile value of the effluent faecal coliform concentration is given as 447 FC per 100ml. The modern design method assures the designer that the effluent faecal coliform concentration will be less than 1000 FC per 100ml with a realistic probability. This allows a much more robust guarantee of the quality of the final effluent for unrestricted crop irrigation even if the critical input design parameters that significantly influence effluent faecal coliform concentration vary by ±20% from their average design values.

Table 6.7 shows that the total land requirement for constructing WSP designed by modern methods based on 50%ile and 95%ile values is higher than the total land required for constructing WSP designed by traditional methods by 38% and 113% respectively. It should be noted, however, that the traditional design method of WSP has been based on deterministic single values of the input design parameters and the procedure of Marais (1974), which are considered by many critics as being unrealistic (Chapters 3 and 4).

Although the traditional design method appears to be use less land, and therefore would seem to be a more economical approach, the final effluent produced by a WSP system based on these design approaches could be unsafe for unrestricted crop irrigation. Using typical input parameters this study has shown that a traditional design procedures would calculate that a

Table 6.7 Comparison of the total land requirement and factor of safety of the traditional design method and modern design method

Design output	Traditional design method	Modern design method based on 50%ile area	Modern design method based on 95%ile area
Area (ha)	25.4	35	54
Factor of safety	1.0	1.4	2.12
No. of maturation ponds	4	7	7
No. of effluent FC concentration/100ml	1000	3	447

model system would need only four maturation ponds to remove faecal coliforms to a concentration of less than 1000 FC per 100 ml. A modified traditional design method has found that five maturation ponds would be adequate to bring the faecal coliform concentration to less than 973 FC per 100 ml. However, the modern design method has indicated that seven maturation ponds would be required to reduce the faecal coliform concentration to less than 447 FC per 100 ml with a 95% probability.

It is observed that 64% and 70% of the total area of WSP designed by modern method based on 50%ile and 95%ile is attributed to the six subsequent maturation ponds whereas the three subsequent maturation ponds designed by the traditional design method require only 38% of the total area.

The factor of safety of WSP designed by the traditional design method is suggested to be “1.0” since it is based on deterministic single values which are assumed to be constant. However, these input design parameters are thought to be variable and are known to be associated with a degree of uncertainty (Chapter 4). Any variation of the critical input design parameters of temperature, population, per capita wastewater flow and BOD which significantly influence the area of WSP could result in failure of the system.

Table 6.7 shows that WSP designed by the modern method have an adequate factor of safety against failure depending on the range values adopted for the input design parameters. If the deterministic input designs parameters as used in traditional design method vary by ±20% from their average design values, the modern design method shows that the resulting WSP have factor of safety of 1.4 and 2.12 based on 50-%ile and 95-%ile values, respectively. The factor of safety is defined as the ratio of the area of WSP designed by using range values of the input design parameters to the area of WSP designed by using the deterministic single values of the input design parameters.

Clearly there are cost implications for adopting a design of a WSP system that guarantees the final effluent will be suitable for the unrestricted irrigation of crops. Table 6.7 shows that for the model input parameters, a WSP system designed by traditional methods requires 25.4 ha of land, whereas a system designed by modern methods requires 35 ha or 54 ha for 50-%ile or 95-%ile values, respectively. The cost of procuring WSP designed by traditional method for the unrestricted crop irrigation is thus relatively cheap, but could expose the public to health risks. On other hand, the cost of procuring WSP designed by modern techniques is higher than the cost of procuring WSP designed by traditional methods. This higher cost associated with modern methods of design is the price that is paid for managing the uncertainties of the input design parameters. Also, by using a realistic model for estimating faecal coliform removal, which has low K_{FC_T} values, more subsequent maturation ponds are needed, contributing to a substantial increase in the overall area of the system.

However, Table 6.6 shows that if the treated wastewater is not to be used for unrestricted crop irrigation, a series of anaerobic, facultative and first maturation pond designed by modern design methods based on 50-%ile value becomes more cost effective than a system designed by traditional methods, reducing the amount of land required by 48% assuming that the input design parameters vary by ±20%.

In summary, this chapter has demonstrated that the modern design method for WSP is efficient in terms of the application of sensitivity analysis, faecal coliform removal, safety and cost. The chapter has shown how sensitivity analysis on the design output of effluent faecal coliform concentration and WSP area is carried out using the Kolmogorov-Smirnov two-sample test. The faecal coliform removal efficiency has been compared between three design methods: the traditional method using the equation of Marais (1974), a modified traditional design method using the empirical equations of Von Sperling (1999) based on a completely mixed model and the equation of Mara (2002), and the modern design method. The area of a WSP system designed by the modern design method and the traditional design method has been compared in terms of the factor of safety and cost of procuring the system.

In conclusion, the modern method for designing WSP is a flexible approach which allows sensitivity analysis to be applied during the design stage. This is something that is not possible when traditional methods for design are used because these methods use deterministic input design parameters. Sensitivity analysis has shown that effluent faecal coliform concentration is significantly influenced by the per capita BOD, per capita wastewater flow, the net evaporation rate, temperature, and the faecal coliform removal constant rate in anaerobic ponds and the dispersion numbers in facultative ponds, assuming that the average values of the input design parameters vary by ±20%. The area of WSP is significantly influenced by the per capita wastewater flow, population, and temperature. The results of the sensitivity analysis indicate that the critical common input design parameters (per capita BOD, per capita wastewater flow, population, temperature, evaporation, faecal coliform removal constant rate in anaerobic ponds) should be determined accurately for the economic design of WSP.

The modern design method for WSP results in a system that is efficient in removing influent faecal coliforms even if the critical input design parameters which significantly influence the effluent faecal coliform concentration vary by ±20%. Traditional design methods can overestimate the faecal coliform removal constant rate such that the resulting maturation ponds are underdesigned.

As a result of the deterministic use of single input design parameters, traditional design methods can be said to produce WSP that have a factor of safety of "1". The modern design method offers a factor of safety of more than "1" depending on the range of the input design parameters and the percentile value adopted.

WSP designed by the traditional method for unrestricted crop irrigation are cheaper to construct than those that use the modern method, but they could expose the public to health risks. The higher costs of WSP designed by modern methods can be attributed to the need to manage uncertainties in the input design parameters and to minimize the risk to the health of the public if the treated effluent is to be of a quality that is sufficiently high to use for unrestricted crop irrigation.

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS FOR FURTHER WORK

7.1 Conclusions

The following conclusions are based upon a critical appraisal of the literature relating to traditional design methods of WSP and the modern methods of designing WSP for unrestricted crop irrigation as presented in this study.

1. Traditional design methods are conservative and unrealistic methods for designing WSP. WSP designed by traditional design methods produce effluent that could pose a risk to public health if the final effluent is to be used for the unrestricted irrigation of crops. The weakness of traditional design methods is the use of input design parameters as deterministic single average values. The input design parameters are subject to uncertainty. These uncertainties can be managed if the design parameters are considered as a range.

2. The completely mixed hydraulic flow regime assumed by Marais (1974) to model faecal coliform removal in WSP is not realized in practice. A dispersed hydraulic flow regime should be used to model the hydraulic performance of WSP since it mimics the non-ideal flow that occurs in WSP.

3. The equation of Marais (1974) for faecal coliform removal, which is currently used for designing maturation ponds, is not recommended for modelling faecal coliform removal in WSP. The model overestimates the faecal coliform removal constant rate. The empirical equations of Von Sperling (1999) are recommended for designing maturation ponds for unrestricted crop irrigation. These empirical equations have been developed based on sound research findings.

4. Modern design methods represent the rational approach to designing WSP. Modern design methods are based on a dispersed hydraulic flow regime that relates more accurately to the real situation in WSP. Modern design methods integrate Monte Carlo simulations, inverse cumulative function of the uniform probability distribution of input design parameters, and the empirical equations developed by Mara and Pearson (1986), Mara (1987), Mara et al. (1997a), Mara (2002) and Von Sperling (1999) to design anaerobic ponds, facultative ponds

and maturation ponds. The use of Monte Carlo simulations enable the design variables to be treated as a range to accommodate uncertainties.

5. The Visual Basic computer program that has been developed is an effective and reliable program for designing WSP due to its ability to present output design data in a statistical format. The statistical output of these design data helps the designer to make decisions about the size of WSP based on available funds and the acceptable health risks. The computer program is user-friendly and is simple to run because of its compatibility with Microsoft Excel Spreadsheet.

6. The rational approach to meeting the safety criteria for unrestricted crop irrigation is to design maturation ponds based on a 95%ile of effluent faecal coliform concentration, which should be less than 1000 FC per 100ml. The modern design method adopts this approach.

7. The modern design method for WSP is flexible and allows sensitivity analysis to be applied during the design stage. Sensitivity analysis has shown that effluent faecal coliform concentration is significantly influenced by the per capita BOD, per capita wastewater flow, net evaporation rate, temperature, faecal coliform removal constant rate in anaerobic ponds and the dispersion numbers in facultative ponds, allowing the average values of the input design parameters to vary by ±20%. Sensitivity analysis has further shown that the area of WSP is significantly influenced by the per capita wastewater flow, per capita BOD, population, and temperature. Sufficient resources should be made available at the design stage to allow these critical parameters to be accurately determined.

8. The modern design method is more efficient in reducing the influent faecal coliform concentration than the traditional design method even if the critical input design parameters which significantly influence the effluent faecal coliform concentration area vary by ±20% from their average design values. The traditional design method overestimates the faecal coliform removal constant rate such that the resulting maturation ponds could be underdesigned.

9. WSP designed by traditional design method can be attributed a factor of safety of unity due to the use of single average values for the input design parameters. On this basis, WSP designed by the modern method have a factor of safety >1, depending on the range of the input design parameters and the percentile value employed. This study has shown that factors of safety of 1.40 and 2.12 are achieved, based on 50-%ile and 95-%ile values, respectively, assuming that the average design values of the input design parameters vary by ±20%.

10. The cost of procuring WSP for unrestricted crop irrigation designed by the modern design method relatively higher than for systems designed by the traditional method. WSP designed by the traditional method use less land and are cheaper to construct. However, this is a result of the use of deterministic single values of the input design parameters, which in turn can result in the underdesign of the maturation ponds, which can expose the public to a health risk.

11. If the treated wastewater is not to be used for unrestricted crop irrigation, a series of anaerobic, facultative and first maturation ponds designed by modern methods based on a 50-%ile value for faecal coliform concentration becomes more cost-effective than the traditional design method. A land saving of 48% can result, assuming that the average input design parameters vary by ±20%.

7.2 Recommendations for further work

It is suggested that the following work needs to be carried out to facilitate the use of modern methods in designing WSP:

1. Determination of the range of the input design parameters range should be researched thoroughly in developing countries. This would allow the extent of uncertainties to be confidently predicted by designers. This could form a rational basis for establishing the range of the input design parameters, rather than using an assumed percentage variation of the average design values to establish the upper and lower end of the proposed range.

2. Pilot-scale WSP designed by modern methods should be constructed to observe the effluent faecal coliform concentration. The pilot-scale ponds should be designed to function hydraulically with dispersed flow by using baffles, and the effluent faecal coliform concentration samples should be measured to validate the proposed design procedure for maturation ponds.

3. The procedures of the sensitivity analysis have been based on varying one input design parameter with the other input design parameters held at a constant value. This does not reflect reality, where all or some of the input design parameters can vary simultaneously. Further research on sensitivity analysis based on variation of the combined input design parameters should be carried out to identify potential worst-case scenarios.

4. Research on the application of Monte Carlo simulations to design WSP for restricted crop irrigation and effluent discharge to receiving water bodies should be carried out. This should validate the concept that Monte Carlo simulation is the safest and most efficient method of managing uncertainty of the input design parameters.

[End of Section 7]

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