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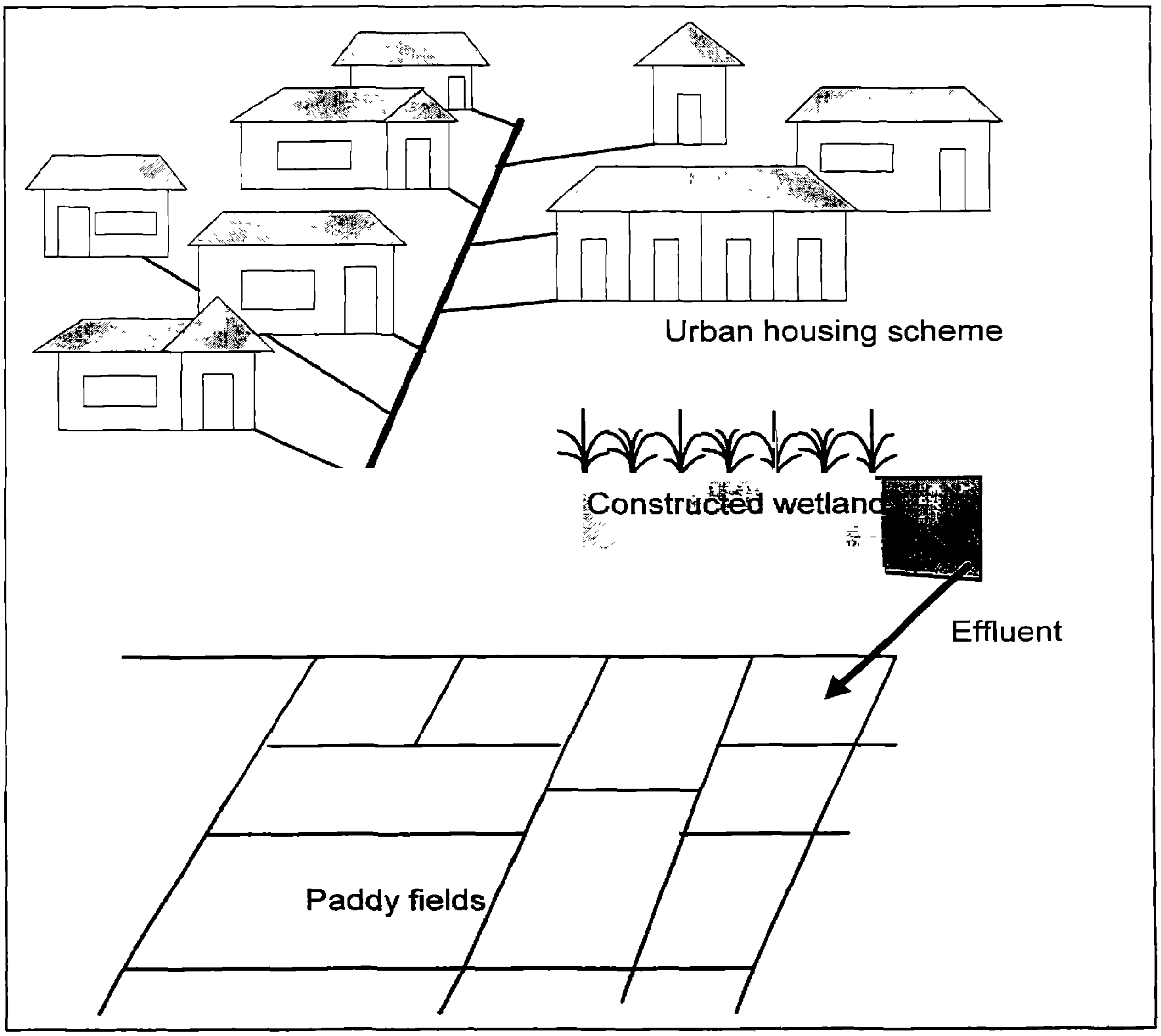

A Strategy for Sanitation in Sri Lanka

Reuse of Wastewater in Irrigation

N.E.M.S.B. Ekanayake

M.Sc. Thesis S.E.E. 102

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A STRATEGY FOR SANITATION IN SRI LANKA

Reuse of wastewater in irrigation

By

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A thesis submitted to the
Department of Sanitary and Environmental Engineering
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ABSTRACT

In Sri Lanka, wastewater collection is available only in the capital, major industrial parks and some major housing schemes. Proposals to provide wastewater collection facilities for provincial capitals under an ongoing urban development programme are available, but there is no key program to upgrade existing sanitation facilities in district and electorate level towns, urban housing schemes, rural housing schemes and villages. High investment and operation and maintenance cost involved with conventional wastewater collection and treatment systems is a major constraint for providing sewerage facilities to these communities. A water shortage during the drought season needs urgent attention.

The objective of this study is to identify appropriate technological options to overcome these problems and to develop a strategy for sanitation and reuse of wastewater in the North Western Province of Sri Lanka. The outcome of this study will be applicable to similar areas in Sri Lanka.

Data were collected to evaluate the existing conditions. A literature survey was done to assess different technologies. A primary selection of technological options was carried out to select suitable options for secondary evaluation. During the secondary evaluation, preliminary designs were completed and cost comparisons were prepared in order to find the most suitable options.

Puttalam town is selected to identify appropriate options for sanitation and reuse of wastewater. The outcome of the study showed that waste stabilisation ponds are the appropriate option for wastewater treatment. The reuse of wastewater can either be used in traditional farmlands or in uncultivated land.

The strategy developed for new urban housing schemes confirmed that provision of sewerage at construction stage of a scheme is feasible and affordable for users. The study also showed that the UASB type of septic tank with constructed wetlands is the most appropriate option for wastewater treatment. The study also developed a strategy to upgrade the sanitation systems in existing urban housing schemes.

On-site sanitation is proposed for small towns, rural housing schemes and villages, which includes pour-flush and VIP latrines.

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Samanmal Ekanayake
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ABBREVIATIONS

FWS	Free Water Surface
GNP	Gross National Product
NGO	Non Governmental Organisation
NWS&DB	National Water Supply and Drainage Board
PE	Population Equivalent
RBC	Rotating Biological Contactor
SF	Subsurface Flow
UASB	Upflow Anaerobic Sludge Blanket
VIP	Ventilated Improved Pit
WC	Water Closet
WHO	World Health Organisation
SOL	Surface Organic Loading Rate

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1. INTRODUCTION

1.1 Background

Sri Lanka is an island situated at 880 kms north of the equator, off the southern tip of the India. The existing population is 19.1 millions and the total land area is 69,600 square kilometres. The island is divided into nine provinces and the area under study is North Western Province. The map of Sri Lanka (showing provincial boundaries and provincial capitals) is presented in Figure 1.1.

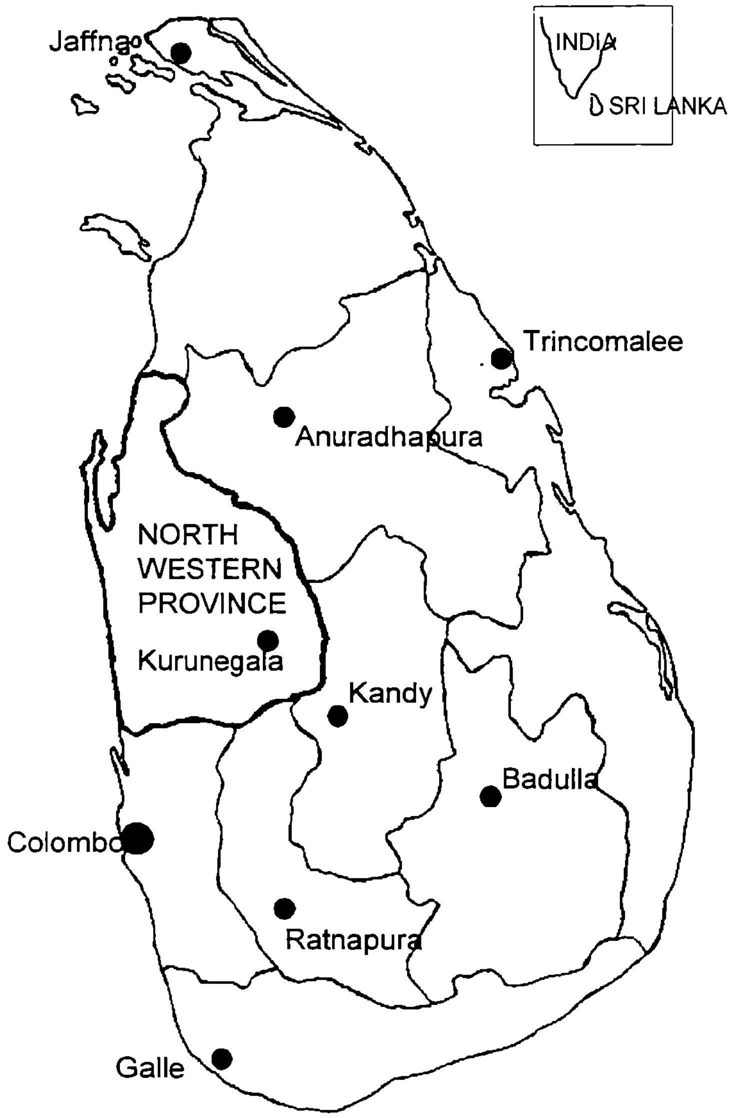

Figure 1.1 Nine provinces and provincial capitals of Sri Lanka

The present wastewater collection facilities cover: the capital, major industrial parks and large housing schemes. There are some proposals to provide wastewater treatment facilities to the other major cities under an ongoing urban development programme.

However there is no key programme to evaluate and upgrade the existing sanitation facilities in the other areas including many district and electorate level towns, housing schemes and villages. The majority of the population in these areas use their own sanitation systems. There is a risk of groundwater pollution as a result of poor sanitation systems in use. High investment and operation and maintenance cost involved with conventional wastewater treatment systems is a major constraint for providing sewerage facilities to these areas.

Therefore it is important to study appropriate low cost sanitation options to improve the present situation. Studying the possibility of reuse of wastewater in the agricultural and domestic applications is an important task too due to the shortage of water during the dry season.

1.2 Objectives

The main objective of this study is to identify appropriate technological options and to develop brief implementation strategies for onsite sanitation, low cost sewage collection, low cost wastewater treatment and reuse of wastewater in the North Western Province of Sri Lanka. The outcome of this study is applicable to similar areas in Sri Lanka.

The specific objectives within the above framework are:

- To identify onsite sanitation options based on population density, income levels, soil conditions and impact on groundwater quality.
- To identify appropriate technology for conveyance and treatment of wastewater by considering cost, population density, income levels and impact on groundwater quality.
- To identify reuse opportunities of wastewater in agricultural and domestic applications.
- To develop a strategy in brief for implementation

1.3 Methodology

The methodology used was as follows:

- Collection of data on population (existing population, growth rate and population densities), existing water supply and sanitation facilities, water consumption pattern, climate, social and economic factors, water quality, health indicators, soil conditions and rates for cost estimation in North Western Province.
- Assessment of present situation using collected data.
- Literature study to consider new technologies.
- Assessment of different technological options and primary selection of appropriate options.
- Secondary evaluation of the appropriate technological options and formulation of a strategy for sanitation and reuse of wastewater in: major towns; urban housing schemes; small towns, rural housing schemes and villages of North Western Province.

1.4 Contents of the report

Chapter 2 contains the existing situation in Sri Lanka regarding climate and topography, social and economic conditions, sanitation facilities and functioning of the present facilities. More detailed information regarding North Western Province is presented.

Chapter 3 comprises the literature study on specific sanitation technologies that may be applied in North Western Province of Sri Lanka.

Chapter 4 explains about assessment of different technological options and primary selection of appropriate options.

Chapter 5 contains the formulation of a strategy for sanitation and reuse of wastewater.

Chapter 6 presents the conclusions and recommendations.

2. EXISTING SITUATION

2.1 Climate and topography

Rainfall divides Sri Lanka into wet and dry zones and an intermediate zone between the two. The mountains and the south-western part of the country known as the "Wet zone" receives ample rainfall (average rainfall of 2,500 mm/year) throughout the year. Most of the south-east, east, northern and portion of the north-western parts of the country comprise the "Dry zone", which receives an average rainfall of 1,500 mm/year mostly during the period of October to January and remains comparatively dry during the rest of the periods. The north-west and south-east coasts receive the least amount of rain 600 to 1,200 mm/year (Abeywickrama *et al.*, 1991). The area under study (North Western Province) consists of all climatic zones. The respective climatic zones are shown in Figure 2.1 (Source-National atlas).

Although the mean annual temperature in the lowlands is around 27 ^{0}C, that of the highlands is 15 ^{0}C. The North Western Province mainly consists of lowlands and the heights vary from 0 to 150 m above mean sea level.

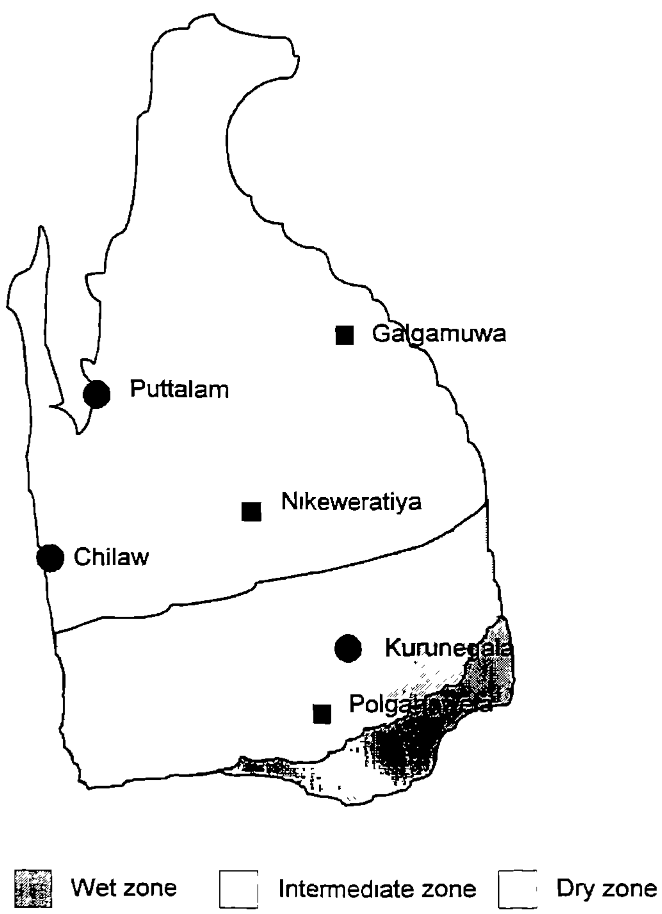

Figure 2.1 Climatic zones in North Western Province

2.2 Social background

Population

The present population in the country 19.1 millions is distributed over 65,606 km^2 land area. About 55% of the total population live in the wet zone and the urban population of the country is 22% of the total population. The population growth rate of the country between 1980 and 1996 was 1.3% and the present growth rate is only 1%. The urban growth rate is 1.62%.

The total population in the North Western Province is 1.7 millions of which 17.1% live in urban areas. The population in the wet and intermediate climatic zones is 48.8%. The existing population in Puttalam town (town selected for case study) is 37,000 and that grows at a rate of around 1.5%.

The population density of the villages varies based on factors such as, availability of infrastructure facilities (electricity, roads, transport etc.), climate, availability of groundwater and distances to nearby schools, hospitals and towns. While the population density of a remote village in the dry zone is around 1 - 2 capita/ha, that of a village closer to a major town goes up to 75 capita/ha.

Housing schemes

Housing schemes can be divided into two major categories as urban housing schemes and rural housing schemes.

Urban housing schemes are formed in the surrounding areas of major towns as a result of urbanisation. Generally the lands closer to the popular towns are divided into smaller pieces ranging from 6 to 15 perches (0.015 to 0.038 ha) and sold to people either by private land developers or by the government. Then the respective people form a housing scheme (a group of houses) by constructing their houses. Present land prices are given in Appendix 2 Table A.2.6.

Rural housing schemes are generally formed as a result of government's housing development programme. The land area per family ranges from 25 to 40 perches (0.06 to 0.1 ha).

2.3 Economy

Economic indicators

The per capita GNP of Sri Lanka is US$ 755 with an annual growth rate of 0.5%. About 60% of the total population categorised as low-income group receives an annual income less than US$ 360/year. About 30% of the population receives between 360 and 1200 US$/year. The high-income group (10%) receives more than 1200 US$/year. The currency used Sri Lankan rupee has a value of Rs 72.60 = 1 US$. The annual inflation of the country is 11.8%. The distribution of the labour force is 48% in agriculture, 21% in industrial sector and 31% in services.

Agriculture

Agriculture is the main source of income for the majority of the population in the country. While tea, rubber and coconut are grown as major export crops; paddy is grown for internal consumption. Only 59.1% of arable lands are cultivated in the island. Irrigation is available only for 29.2% of the cropland. Artificial fertiliser is generally used for crop cultivation. Tea, rubber, coconut and paddy are major crops, which consume artificial fertilisers. The average fertiliser use in the country is 2,270 kg/ha of arable land per year.

Paddy and coconut are the major crops grown in North Western province. The percentages of land areas utilised by the different crops in North Western Province are shown in Figure 2.2.

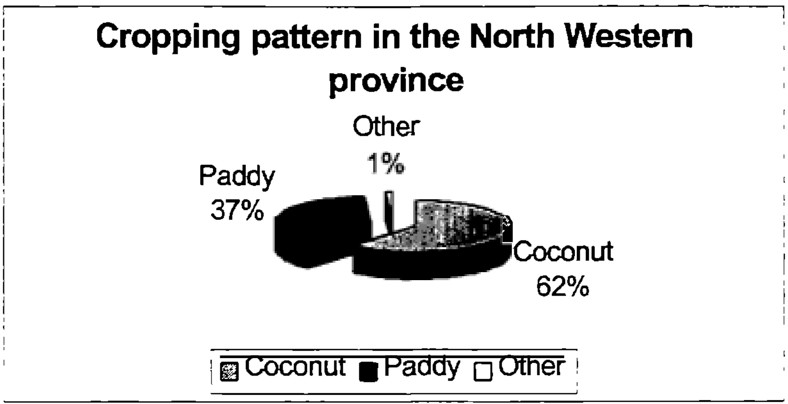

Figure 2.2 Land area utilisation by different crops

Cultivation of the seasonal crops like paddy, vegetables etc. depends on availability of water. These crops are cultivated twice a year during the two cultivation seasons called "Yala" and "Maha" in areas where water can be drawn from irrigation schemes or sufficient rainwater is available. Crop cultivation is restricted to one season in areas where irrigation water is not available and rainfall is inadequate for cultivation in the two seasons. Even during this single cultivation season there is a risk of crop failures due to uncertainty of the rainfall pattern.

Availability of surface water
The annual discharge of the rivers in the island is 47,000 millions m^3/year. The runoff through the dry zone is 18,000 millions m^3/year. Although several irrigation schemes have been constructed to utilise this runoff, the total demand is not met.

The total average yield of surface water in the dry zone of the North Western Province is 835 millions m^3/year. This is not adequate to meet the present agricultural demand of 2,212 millions m^3/year. Even there is a proposal to divert "Kelani" river to North Western Province, this is not implemented due to very high capital cost.

2.4 Sanitation

Water supply coverage
The percentage coverage by piped water supply in the island is around 32%. The present supply rate (through water supply schemes) is 182 million m^3/year.

The percentage coverage by piped water supply in the North Western Province is 15.7%. However the percentage coverage in the urban areas is generally high. The details of the water supply facilities in the townships in North Western Province are presented in Table 2.1.

Table 2.1 Water supply coverage of the urban centres in North Western Province

Category	No of towns	Towns with water supply	Total population
Major townships (Population >25,000)	3	3	124,000
Other townships (Population 3,000 - 25,000)	29	25	167,000

Source: NWS&DB – North Western Region

Water sources

People use shallow wells, deep wells (mainly in the dry zone) and surface water as their sources in areas where piped water is not available. Many people use the individual shallow well. Deep tube wells fitted with hand pumps are popular in the rural areas of dry zone. Surface water sources are generally used for bathing and washing cloths. But few people still use surface water for drinking.

Water consumption pattern

The average per capita water consumption of the served population in the different areas of the island varies between 100 – 177 L/c/d. The respective figure for stand post user varies between 30 – 40 L/c/d (the details are presented in Appendix 2 Tables A.2.1 and A.2.2). The average per capita consumption in the North Western province is 135 L/c/d and that of a stand post user is 38 L/c/d. The consumption pattern of the people with their own individual piped water supplies is almost the same as that of the water connection users. The average water consumption and population served in water supply schemes in the area is presented in Appendix 2 Table A.2.3.

Revenue collection

The revenue collection from the water consumers is satisfactory. According to the National Water Supply and Drainage Board the present billing and collection ratio is 85%. The respective figure in the North Western Region is 82%.

Wastewater treatment

Wastewater treatment facilities are available only for few housing schemes and major industrial parks in the country. The Colombo Municipal Council area (Capital of the country) with an estimated population of 825,000 is the only municipality with piped sewerage. None of Colombo's sewage is treated before discharge. The Government is planning to provide sewage treatment facilities for some provincial capitals under the present urban development programme, but this programme does not cover any other community or town.

Sanitation in the major towns

The people in the towns use their own sanitation systems, such as latrines with cesspools or septic tanks. Most of these systems are not up to the required standard and therefore imply a health risk. Wastewater is directly discharged to existing surface water drains in some densely populated city centres. Even when the piped water is available, some people still use shallow wells. Sometimes these wells are located too close to a septic tank.

Sanitation in the urban housing schemes

The urban housing schemes are located a few kilometres outside the town limits. Piped water supply is not available at the sites. The occupants, who are generally of middle (or high) income category, construct their own water supply system that generally consists of a shallow well as the source, a pump, an overhead tank and a piping system. A flush or pour flush toilet connected to a septic tank is the most common waste disposal method used in these houses. The shallow wells and the septic tanks are too close and sometimes the septic tanks are constructed upstream of the shallow wells not considering direction of the groundwater flow. Therefore the possible risk of groundwater pollution is higher than that of the towns, villages and rural housing schemes.

Sanitation in the small towns, villages and rural housing schemes

The majority of the population in this category use their own sanitation systems such as latrines with cesspools or septic tanks. Most of these systems are not up to the required standards. Many people do not follow any guideline when locating their cesspools. As a result many cesspools are located too close to a drinking water source (shallow well).

There are some people in the rural areas without any sanitary facilities. Although the Government and some non-governmental organisations (NGO's) have launched several sanitation programmes in these areas, some areas yet have to be covered. About 25% of the households are lacking sanitary toilet facilities.

Groundwater quality

The records on water born diseases are good indicators for groundwater pollution. The number of water born diseases reported in the North Western province (Kurunegala and Puttalam districts) is presented in Table 2.2.

Table 2.2 Water born diseases reported from North Western province

Disease	Number reported						
	Kurunegala district					Puttalam district	
	1995	**1996**	**1997**	**1998**	**1999**	**1998**	**1999**
Dysentery	953	942	2071	1844	726	338	379
Typhoid and Paratyphoid	303	252	360	519	225	0	12
Viral Hepatitis	567	275	488	563	196	0	166
Cholera	0	0	86	86	20	383	58

Source: RDHS offices Kurunegala and Puttalam

Unlike in the town water supplies and deep wells (maintained by NWS&DB), the bacteriological and chemical quality in the shallow wells is not regularly checked. Therefore only few data are available. The information about bacteriological and chemical quality in shallow wells in the area is presented in Appendix 2 Tables A.2.4 and A.2.5.

3 LITERATURE STUDY

3.1 Previous studies at IHE

Relevant MSc studies carried out on sanitation and reuse of wastewater in agriculture are briefly described hereafter. M. Incencio Sousa from Cape Verde, Jamal Mustafa from Palestine and V.F. Awananto from Indonesia carried out these MSc studies.

Sousa formulated "A strategy for sanitation in Cape Verde". Under this topic he discussed the problems of scarcity of water resources, increased use of desalinated water for domestic purposes and reuse of wastewater in irrigation to overcome the problem of salty ground water and food production (about 90% of the food has to be imported). He proposed small bore sewers with septic tanks for the conveyance of wastewater and sand filtration with duckweed cultivation for the treatment; slow sand filtration for unrestricted irrigation and duckweed for removal of excess nutrients.

His proposal was focused on centralised wastewater treatment consisting of anaerobic ponds and sand filter beds. It may be considered as an option for wastewater treatment in major towns in North Western Province of Sri Lanka. However the appropriate treatment options for Sri Lanka may be different, as the local conditions in Cape Verde are different from those of Sri Lanka.

The topic of Mustafa is "Onsite wastewater treatment, disposal and reuse". The main objective of his study is to develop a trickling filter for onsite operation in order to treat and reuse grey water. The conditions like household size, electricity cost and income levels etc. in Palestine are different from those in Sri Lanka. However, the proposed option may also be suitable for Sri Lanka. He also formulated a strategy for onsite treatment, disposal and reuse of wastewater in Palestine. To prevent groundwater pollution, Mustafa proposes cartage of black wastewater to be dried in a lagoon and reuse in agriculture.

"Appropriate sanitation systems for Indonesian urban areas" is the topic of Awananto. Awananto, who discussed low cost sanitation options suitable for the conditions in Indonesia. He considered individual and shared disposal systems as well as communal disposal systems of special interest with the connection of several houses (cistern flush, pour flush) to a simplified "USAB reactor". Awananto's study is based on conditions suitable for Indonesia and which are different from those in Sri Lanka, in particular with regard to the level and kind of community involvement.

3.2 Wastewater treatment and reuse

3.2.1 Effluent quality guidelines

WHO (1989) recommended microbiological quality guidelines for wastewater reuse in agriculture. These guidelines are categorised based on reuse condition and exposed group (Pescod, 1992). The respective guidelines are presented in Table 3.1.

Table 3.1 Recommended microbiological guidelines for wastewater use in agriculture

Group	Reuse condition	Exposed Group	Intestinal Nematodes (Arithmetic mean no. of eggs /litre)	Faecal coliforms (geometric mean) no./100 ml	Wastewater treatment required to achieve the required microbiological quality
A	Irrigation of crops likely to be consumed uncooked	Workers Consumers Public	<1	<1,000	A series of stabilisation ponds to achieve the microbiological quality indicated or equivalent treatment
B	Irrigation of cereal crops, industrial crops, pasture and trees	Workers	<1	No standard	Retention in stabilisation ponds for 8 – 10 days or equal helminth and faecal coliform removal
C	Localised irrigation of crops in category B if exposure of workers and public does not occur	None	Not applicable	Not applicable	Pre – treatment as required by irrigation technology, but not less than primary sedimentation

Source: Pescod M.B., 1992

3.2.2 Primary treatment

Shared and communal septic tanks

Anaerobic pre-treatment at on-site level for a number of households ("shared" treatment) or for a township ("communal"), followed by transportation through cheaper shallow sewers or open drains up to a central location outside the city for final treatment and disposal is an appropriate and cost effective system for developing countries. While the shared facility is designed for 35-75 PE the communal facility can go up to 150-800 PE (Alaerts *et al.*, 1993).

Small scale UASB reactors

A field experiment carried out in Bandung/Indonesia showed satisfactory results for two UASB reactors tested for black water and for combined black and grey water treatment. The respective results are presented in Table 3.2.

Table 3.2 Experimental results for black and combined wastewater treatment

	Black wastewater	Combined wastewater
Reactor volume (liters)	860	860
No. of houses connected	2	2
HRT (h)	16	1
Sludge accumulation rate (kg/d)	0.0397	0.0758
Amount of seed sludge required (kg)	3.552	7.185
% Removal		
CODtot	89 - 93	67 - 75
CODfil	69 - 81	22 - 47
BODtot	86 - 95	46 - 82
TSS	93 - 97	74 - 81

Source. UASB research project final report – Indonesia, 1988

3.2.3 Post treatment

(a) Sand fitration

This is one of the oldest techniques used in wastewater treatment and applied for various cities in Western Europe 100 years ago. The technique was first used 2000 years ago in the city of Athens, where the ancient Greeks used to spread their wastewater over a sandy area (Guillotau *et al.*, 1993).

In the process of filtration and percolation, the effluent is poured over a sand layer. The operation is intermittent to prevent algal growth. During percolation there are aerobic degradation of organic matter at a very low biological load, supply of oxygen through pores (aeration), nitrification and denitrification.

Experimental study at Ben Sergo, Agadir, Morocco

Guessab *et al,* (1993) analysed the treatment efficiency of an experimental plant consisting of an anaerobic lagoon (volume 1,500 m^3) and 5 infiltration basins (surface area 1,500 m^2 each) at Agadir in Morocco. The construction of this plant was based on the results observed from a pilot plant with a square area 300 m^2. The purification plant receives wastewater at a rate of 1,050 m^3/d from 20,000 population equivalents. The 2 m thick sand basins receive pre-settled wastewater by batches. Three 350 m^3 batches are discharged into 3 separate basins during 3 consecutive days and then left to rest for 2 days before a new cycle starts. The average COD concentration of raw wastewater was 1,189 mg/l. The bacteriological, parasitalogical and physiochemical removal efficiencies were also determined in this experimental plant.

Excellent bacteriological removal efficiencies are observed from the results. The relative removal efficiencies of faecal coliform and faecal streptococci are 99.99% and 99.999% respectively. Table 3.3 shows the respective results.

Table 3.3 Bacteriological removal efficiencies of Agadir treatment plant

	Geometric mean of the bacterial concentration (number/100 ml of water)	
	Raw water	Treated water
Faecal coliforms	*62 $\times 10^5$	328
Faecal steptococci	*207 $\times 10^5$	346

* Confidence interval at 95% Source: Guessab *et al,* (1993)

The parasitological removal efficiency is also well within the expected limit. The 100% removal efficiency of Helminth eggs satisfies the strict WHO guideline for wastewater reuse in agriculture (1 No/L). Table 3.4 shows the respective results.

Table 3.4 Parasitological removal efficiencies of Agadir treatment plant

	Parasitological count (number of eggs/1 litre of water)	
	Raw water	Treated water
Helminth eggs	*214	0
Nematodes eggs	139	0
Cestodes eggs	75	0

* Confidence interval at 95% Source: Guessab *et al,* (1993)

The physiochemical removal ratios are also very satisfactory. The removal efficiencies of TSS, COD and TKN are 99%, 96% and 85% respectively. There is a significant increase of NO_3^{2-} due to nitrification (from 0 to 146 mg/l). There is no denitrification; sufficient air is available for degradation of organic matter.

Loading rates applied in the infiltration basins at Agadir

In the Agadir wastewater treatment plant, the quality of the raw wastewater has been compared with the final effluent Therefore, the influent COD concentration to the sand filters is calculated by assuming the COD removal efficiency of the anaerobic lagoon in order to assess the organic loading rates to these basins.

The volumetric organic loading rate to the anaerobic pond is 832 g COD/m^3.d while the hydraulic retention time is 1.4 days. The applicable volumetric organic loading rate and the expected BOD_5 removal efficiency of an anaerobic lagoon in a warm country (minimum monthly air temperature > 20 ^{0}C) are 300 g BOD_5/m^3.d and 60% respectively (Veenstra *et al.*, 1998). Assuming the COD/BOD ratio of raw wastewater as 2, the respective applicable organic loading rate becomes 600 g COD/m^3.d. At the volumetric organic loading rate of 832 g of COD/m^3.d, a COD removal efficiency between 45 - 55% can be safely expected for the anaerobic lagoon, since Morocco is a fairly warm country. Accordingly the expected influent COD concentration to the sand filtration basins is between 535 - 654 mg/l. Using these values the respective surface and volumetric organic loading rates of the infiltration basins are calculated hereafter in section (c) " Dimensioning of the filter beds".

Experimental study at Saint Symphorien de Lay, France

Guilloteau (1993) obtained satisfactory results from an experiment carried out to assess the performance of the Saint Symphorien de Lay wastewater treatment plant (France) consisting of 2 primary settling tanks and 2 filtration basins (Guilloteau *et al.*, 1993). The plant received wastewater at a rate 70 to 75 m^3/d from 500 population equivalents. The average COD concentration of the raw wastewater was between 412 to 620 mg/l during the experimental period of one month. The results of the experiment showed significant removal efficiencies of 92% and 97% for TSS and COD respectively.

Dimensioning of the filter beds

The operating cycle of an infiltration basin as Agadir has two phases namely loading or flooding phase and resting or drying phase (see Figure 3.1). During the flooding phase the wastewater is fed in batches to the filtration basins. In between two batches there is a certain rest period for the sand beds. The resting periods are needed for aeration (natural supply of air to the pores) and for full microbiological degradation of organic matter. It is sensible to consider a full cycle when considering plant efficiency.

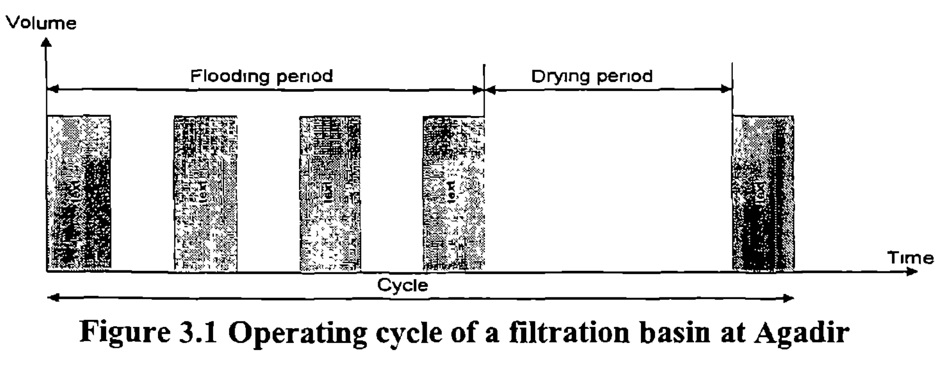

Figure 3.1 Operating cycle of a filtration basin at Agadir

The comparisons of the surface and volumetric organic loading rates (SOL and VOL) of 3 purification plants are compared in Table 3.5.

Table 3.5 Organic loading rates (based on full cycle of flooding and resting)

Purification plant	Surface area m^2	Sand depth m	Flooding period days	Drying period days	Influent COD mg/l	Effluent COD mg/l	SOL g.COD $/m^2.d$	VOL g.COD $/m^3.d$
Ben Sergo – Moroco[1]	1500	2	3	2	*535 -654	52	38 - 46	76 - 92
St Symphorien – France[2]	225	1.7	7	7	368	55	17	30
Limoge – France[3]	340	0.6	3	3	153	75	36	22

Sources 1. Guessab *et al.*, 1993 *Assumed based on pre-treatment efficiency (anaerobic pond)
2. Guilloteau *et al.*, 1993
3. Brissaud *et al.*, 1993

The respective hydraulic loading rates are presented in Table 3.6.

Table 3.6 Hydraulic loading rates (based on full cycle of flooding and resting)

Purification plant	Capita	Flow m^3/d	Hydraulic loading $L/m^2.d$
Ben Sergo - Morocco	20,000	350	140
St. Symphorien - France	500	72.5	80
Limogne - France	450	68	75

Sources 1. Guessab *et al.*, 1993
2. Guilloteau *et al.*, 1993
3. Brissaud *et al.*, 1993

Danish experience

Nielsen *et al.*, (1993) summarise the results and experience on sand filtration systems that have been collected since the beginning of the use of biological sand filter systems in Denmark in the late 70's. The organic surface loading rates (SOL) used in the Danish systems are 4 - 18 g of $BOD_5/m^2.d$ and the respective removal efficiencies are between 90 - 95%.

Characteristics of the filter media

Coarse sand media are generally used in the filter basins. The characteristics of the sand media used in France and Denmark are presented in Table 3.7.

Table 3.7 Characteristics of the sand media's used in the wastewater treatment

Location	Effective size d_{10} (mm)	Mean grain size d_{60} (mm)	Uniformity coefficient' d_{60}/d_{10}
France	0.1 - 1	0 2 - 2	2
Denmark	0.5 - 1	1 - 2	<3.5

Multi layered filter models

Arto Latvala., (1993) of Finland has proposed multi layered intermittent sand filters as an alternative to the conventional infiltration basins for wastewater treatment. The aim of that study was to develop more compact filter to save the required land area. They introduced settled wastewater to multi layered filter models at loading rates between 180 to 450 litres/m^2.d. The mean value of the BOD removal efficiency achieved (including separate primary settling) in this experimental study was 86%.

Typical arrangement of a filter basin

A cross section of a typical sand filtration basin is shown in Figure 3.2. The influent wastewater is evenly distributed on the sand surface using inlet distribution pipes. The drainage pipes laid in a pebble media collect the filtered effluent. A plastic membrane or clay generally seals the bottom.

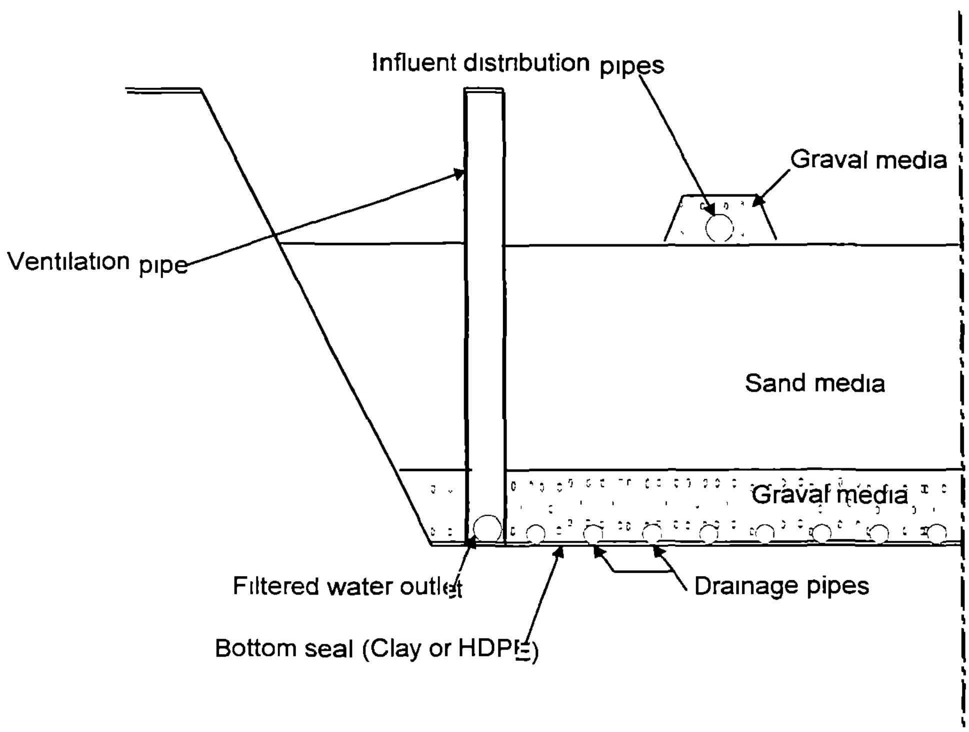

Figure 3.2 Cross section of a typical filtration basin

Operation problems in filter basins

F. Brissaud and J. Lesavre (1993) have discussed 10 years experiences about the infiltration and percolation process in France. They analysed the performance and operational problems of 7 filtration plants of population equivalents ranging from 400 to 1,700. Non uniform spreading of the influent, short circuiting and short detention times were the major problems identified by them. They have also discussed the remedial measures to be taken to overcome these problems and to achieve a higher microbiological quality. These proposals can be briefly listed as;

- Fractionating the inflow
- Better utilisation of the infiltration area
- Elimination of short circuiting
- Proper utilisation of depth of the filter bed for removal of bacteriological pollution

(b) Pond systems

Waste stabilisation ponds (algae ponds)
A waste stabilisation pond is an attractive method for wastewater treatment and reuse in rural areas. It is an attractive treatment method for the areas where land prices are low, climate is warm and facilities for operation and maintenance are poor. As a result of higher removal efficiencies, the World Bank has strongly recommended waste stabilisation pond effluent for agricultural use (Pescod., 1992). The high nutrient value in algae is an added advantage when used as a fertiliser.

The mosquito breeding in the water can be effectively controlled by stocking fish that feed on mosquito larvae (Polprasert., 1996).

Comparison between microphytic and macrophytic ponds
L. Mandi (1993) compared the performance of waste stabilisation ponds (algae ponds) and macrophyte ponds (wetlands) under same loading rate and climate. Both systems received domestic raw effluent during the experimental period. According to the results of this experiment COD and TSS removal efficiencies in the macrophyte ponds were better than those of algae ponds. However the pathogen (faecal coliform and faecal streptococci removal was better in the microphytic ponds. No helminth eggs were found from the effluent of both systems. The results obtained during the summertime are given in Table 3.8.

Table 3.8 Removal efficiencies of macrophyte and algae ponds

	Percentage Removal	
	Macrophytic pond	Microphytic pond
COD	87*	45*
TSS	95*	42*
$NH4^{+}$	63*	72*
PO^{4}_{2-}	43*	78*
Faecal Coliform	96.2	99.6
Faecal Streptococci	94 7	99.7

*During summertime

The microphytic ponds showed thus favourable conditions for agricultural reuse. The water losses due to evapotranspiration was very high (60%) in the macrophytic system.

Duckweed ponds
There are several advantages in using duckweed for wastewater treatment. Some of them are:

- Efficient nutrient uptake from wastewater
- Tolerate high concentrations of detergent and a number of toxic substances
- Absorption of heavy metals
- High protein content
- Odour control
- Harvesting is less complicated than water hyacinth

The BOD removal is very effective in duckweed ponds. Removal efficiencies of 95-99% have been reported. BOD loading rates of between 100 and 160 kg/ha.d is suitable to obtain an effluent quality of 30 mg BOD/l or less. Very high pathogen removal has been reported in duckweed ponds One field experiment showed faecal coliform reduction from 4.57×10^4/ml in the influent to values below 100/ml in the effluent (Duckweed research project report, 1997).

Evapotranspiration
Evapotranspiration losses may comparatively be high in macrophytic ponds. But duckweed has a positive effect on reducing water losses. The positive contribution of duckweed appears to be larger in cases where high evaporation occurs due to dry air, wind etc. (Duckweed Research Project report, 1997).

Mosquito control
Two conflicting opinions on mosquito control in duckweed ponds exist: duckweed ponds enhance or prevent mosquito development.

Polprasert indicates that aquatic vegetation including duckweed enhance mosquito population. The vegetation increases the mosquito population by protecting larvae from wave action, providing a habitat for breeding and interfering with control procedures. The major vectors are *Anopheles*, transmitting malaria and *Mansonia*, which carry filariasis and encephalitis. The eggs of *Mansonia* are laid at the undersides of leaves of aquatic weeds just above the water surface. Then the mosquito larva inserts its respiratory siphon into the air-containing tissues of the plants and need not to come to the water surface for air. The air is consumed from the submerged portions of the plants, especially from the roots. Different *Mansonia* species have preferences for different plants. The water lettuce seems to be the most common host, followed by water hyacinth, *Azolla* and duckweed (Polprasert., 1996).

The Duckweed Research Project report (1997) explains positive effects of duckweed on mosquito control. According to the report, placing of larvae in water with a dense cover of *Wolffia* resulted in dying of insects within several hours. A positive effect on duckweed cover on the decrease of mosquito larvae has been reported for various duckweed species, including *S.punctata*, *L.minor*, *Wolffia* and *Spirodala*. It also explains about positive toxic effects of duckweed on larvae. The report also suggests future research on mosquito control by duckweed.

(c) Constructed wetlands
Constructed wetlands are man-made systems to simulate treatment of wastewater in natural wetlands. Two types of constructed wetland systems exist namely, free water surface (FWS) and subsurface flow (SF). In FWS system wastewater flow is kept 0.1 – 0.6 m. above the soil surface (in parallel basins or channels) and in SF system the water depth is maintained just below the soil surface.

Macrophytes have a very important role for the removal of pollutants in constructed wetlands. They provide a surface for microbial growth. The oxygen transfer by the aquatic plants in to rhizosphere increases treatment efficiency.

BOD and TSS removal
BOD and TSS removal in wetland systems is very effective as presented in Table 3.9. These results are comparable with findings in other plants.

Table 3.9 Removal of BOD and TSS in wetlands

PROJECT	Flow m^3/d	Wetland type	BOD (mg/l)		SS (mg/l)		% reduction	
			In	Out	In	Out	BOD	SS
Lisstowel, Ontario	17	FWS	56	10	111	8	82	93
Santee, CA	N.A	SF	118	30	57	5.5	75	90
Sydney, Australia	240	SF	33	4.6	57	4.5	86	92
Arcata, CA	11,350	FWS	36	13	43	31	64	28
Emmitsburg, MD	132	SF	62	18	30	8.3	71	73
Gustine, CA	3,785	FWS	150	24	140	19	84	86

Source. Polprasert., 1996

Schierup and Brix (1990) have monitored many horizontal flow systems in Denmark since 1980. Many of these systems have soil based media. The removal efficiencies of BOD5, TSS, total N and total P are presented in Table 3.10.

Table 3.10 Removal efficiency of subsurface flow systems – Danish experience

Parameter (mg/l)	Number of samples	Influent mean ± SD	Effluent mean ± SD	% Removal
BOD_5	77	97.0 ± 81.0	13.1 ± 12.6	86
TSS	80	98.6 ± 81.6	13.6 ± 11.1	86
Total N	73	28.5 ± 14 7	18.0 ± 10.7	37
Total P	67	8.6 ± 4.5	6.3 ± 3.5	27

SD = Standard deviation Source: Cooper *et al.*, 1996

The removal of coliforms in both FWS and SF wetlands in US and Canada in summer season is presented in Table 3.11.

Table 3.11 Pathogen removal in constructed wetlands (USA and Canada)

Location	Summer season		
	Influent	Effuent	% reduction
Santee, CA (bullrush)[a]			
Total coliform, no /100 ml	$6.5*10^7$	$3*10^5$	99.54
Bacteriopage, PFU/ml	2,300	26	98.87
Iselin, PA (cattalis & grass)[b]			
Faecal coliform no./100 ml	$1.0*10^6$	723	99.93
Arcata, CA (bullrush)[c]			
Faecal coliform no./100 ml	1,800	80	95.56
Listowel, Ont. (cattalis)[c]			
Faecal coliform no./100 ml	198,000	400	99.8

[a] Gravel bed, subsurface flow Source: Veenstra., 1997
[b] Sand bed, subsurface flow
[c] Free water surface

According to a dry weather survey carried out at Leek Wotton Treatment systems (UK) during a period of five days, the overall mean of the percentage removal of total coliforms was 97.1 while that of the *E.coli* was 97.3 (Cooper *et al.*, 1996).

A monitoring programme carried out by Bavor in New South Wales, Australia showed that standard wetlands systems were very effective in removing faecal coliform up to a standard of

200 F.C/100 ml (Bavor., 1994). This standard is well above the WHO of 1,000 F.C/100 ml set for reuse for irrigation of crops likely to be consumed uncooked.

Mode of operation
Generally subsurface flow systems are designed for parallel operation. However Cooper describes Midleton (Shropshire) sub subsurface flow wetland system which deals with considerably low wastewater flows and was initially set up with two beds in parallel but was later switched to series operation. It has two beds 8 m wide and 10.5 m long with a depth ranging from 0.60 m at the inlet to 0.705 m at the outlet. The area used is 5.6 m^2/PE. The respective results are presented in Table 3.12.

Table 3.12 BOD_5 l and TSS removal at Middleton wetland system

	BOD (mg/l)			TSS (mg/l)		
Parallel operation in 1992	Feed	Effluent		Feed	Effluent	
	306	46		105	25	
Series operation	Feed	Bed 1	Bed 2	Feed	Bed 1	Bed 2
in 1993	333	109	22	109	43	15

Source. Cooper *et al*, 1996

Loading rates
Cooper describes the mass loading rates applied in the Danish subsurface flow systems. Table 3.13 shows the respective results.

Table 3.13 Loading rates applied in the Danish systems

	No of systems tested	Mass loading $g/m^2.d$ (mean + SD)
BOD5	56	4.8 ± 5.97
TSS	51	5.22 ± 7.37
Total N	57	1.15 ± 0.79
Total P	50	0.33 ± 0.27

SD = Standard deviation Source: Cooper *et al.*, 1996

Media selection
Generally medium to gravely sand is used in the subsurface flow systems. The media characteristics of the sand are given in Table 3.14.

Table 3.14 Media characteristics of the subsurface flow systems

Media type	Max.10% grain size mm	Porosity (n)	Permeability (k_s) $m^3/m^2.d$	k_{20} (Rate constant for BOD removal)
Medium sand	1	0.42	420	1.84
Coarse sand	2	0.39	480	1.35
Gravely sand	8	0.35	500	0.86

Source: Veenstra., 1997

Aquatic plants in the wetland bed
There are many varieties of aquatic plants, which have ability to treat wastewater. The main aquatic macrophytes used in the subsurface flow constructed wetlands include *Phragmites australis* (the common Reed), *Typha latifolia* (the Reed Maces), *Juncus effusus* and *Conglomeratus* (the Rushes), *Schoenoplectus lacustris* (the true Bulrush) and *Carex* (Sedge

family). *Phragmites australis* is one of most productive, widespread and variable wetland species in the world. The multiplicity of biotypes confers a wide tolerance of climatic conditions (Cooper *et al.*, 1996). The aquatic plant types and the characteristics of these plants are presented in Table 3.15.

Table 3.15 Macrophytes in subsurface flow wetlands

Emergent species	Temperature ^0C		Optimum pH
	Desirable	Survival	
Phragmites	12 - 33	10 - 30	2.0 – 10.0
Typha	10 - 30	12 - 24	4.0 – 10.0
Juncus	16 - 26	-	5.0 – 7.5
Schoenoplectus	16 - 27	-	4.0 – 9.0
Carex	14 - 32	-	5.0 – 7.5

Source: Cooper *et al*, 1996

Mosquito breeding

The mosquito-breeding problem in the FWS type constructed wetlands is almost similar to that of the macrophytic ponds, which was described previously. However this problem is not related to the SF type constructed wetlands, as the water surface is kept below the soil surface.

(d) Land treatment

The application of wastewater at a low loading rate is widely used for agriculture. The prime advantages of this process are;

- Low cost treatment of wastewater
- Conservation of water through irrigation
- Recovery of the nutrients for agriculture

The application of wastewater must be controlled effectively to get the desired efficiency and to prevent groundwater contamination. The application rate varies from 2.5 to 10 cm/week depending on the type of the crop and soil. A schedule should be maintained for optimum application of wastewater depending on the climate and BOD loading. A high effluent quality can be expected from the land treatment process as shown in Table 3.16.

Table 3.16 Expected effluent quality from land treatment

Constituent	**Average**	**Upper range**
BOD (mg/l)	<2	<5
TSS (mg/l)	<1	<5
Ammonia nitrogen as N (mg/l)	<0.5	<2
Total nitrogen as N (mg/l)	3*	<8*
Total phosphorus as P	<0.1	<0.3
Faecal coliform, (no./100 ml)	0	<10

*Concentration depends on loading rate and crop

4 TECHNOLOGICAL OPTIONS

4.1 Technological options for on-site sanitation

Sanitation is one of the prime objectives of improving health standards of the people in the developing countries. On-site sanitation is one of the cheapest and sustainable options in achieving this objective. The widely spreaded on-site sanitation options and their applicability and limitations are discussed in this section.

Pit and pour-flush latrine and cistern flush with septic tanks are common types of on-site sanitation systems in Sri Lanka. However pour-flush is widely spreaded all over the country. It is popular among middle-income and low-income people due to moderate cost, convenience and low water requirement. Generally the low-income groups use simple pit latrines and to a small extent VIP latrines. The high-income and some middle-income people use cistern flush with septic tanks.

The pour-flush latrine is an appropriate method. The squatting pans are fabricated locally (out of cement) and people can afford the price.

The ventilated pit latrine can also be considered as an appropriate method in rural areas where the pit can be kept at a safe distance from the water source. The pit is ventilated and fly control is provided through a screen at the top of the ventilation pipe. It is very attractive among low-income people due to very low cost.

The water closet is an attractive system among the high-income people. It is becoming popular among the urban population, but the high water requirement is a major disadvantage in this system.

Bucket latrines, which had been in use till late 70's are unhygienic and therefore rejected by the society.

The Aqua Privy is more expensive than the pour-flush system and it functions unsatisfactorily because of leakage. The water seal is lost if insufficient water is added.

Introduction of dry sanitation is not practicable in Sri Lanka since the society has a general attitude that water should be used after defecation. Therefore composting toilets are not an appropriate method. Its complicated operation is another reason for rejection.

Separation of grey and black water is practised in the existing systems. The grey wastewater produced from bathrooms and kitchen is directly discharged on ground and only black wastewater is sent to septic tank. Therefore reuse of grey water for the activities like toilet flushing and gardening can be considered.

Appropriate on-site sanitation options

The pour-flush latrine, VIP latrine and water closet with septic tank are considered as most appropriate on-site sanitation options. The advantages and disadvantages of these options are presented in Table 4.1.

Table 4.1 Advantages and disadvantages of selected on-site sanitation options

Sanitation option	Advantages	Disadvantages
Pour- flush latrine	Low cost Convenient for the user Pit is not visible Less water use Toilet can be constructed within the house No odour problems	Expensive than VIP latrine
Ventilated pit latrine	Low cost Water not required Easy construction Less odour problems Control of flies	Additional cost for vent pipe Need to keep the inside dark
WC with septic tank	Convenient for the user Pit is not visible Toilet can be constructed within the house No odour problems	Expensive High water requirement

4.2 Technological options for off-site sanitation

The common off-site sanitation options and their applicability and limitations are discussed and primary selection is done in this section.

4.2.1 Wastewater collection

Technological options for wastewater collection

The options available are: combined and separate conventional collection systems, shallow small bore sewers and intermediate systems with septic tanks or interceptors and small-bore sewers.

Combined sewers are far too expensive because of high intensity rainfalls. Overland flow of storm water is practised. The conventional (separate) sanitary sewer system is preferred. The maintenance of conventional sewers is much less than the small-bore sewers. High capital cost is the main disadvantage.

Small-bore sewer is another option to transport sewage either to a treatment site or to a trunk sewer. The sewers are designed to be flushed frequently and the sludge is flushed along by successive waves of wastewater Densely populated areas offer favourable conditions for such operations. But frequent blockages can be expected when wastewater production is low and troublesome solids (polythene, cloths etc.) are disposed.

Small-bore sewers can be preceded by on-site septic tanks. In this system, primary settling and stabilisation of sludge is done in the septic tank. Therefore timely removal of sludge is required to prevent blockages in the sewer network, which was never done in practise. Perfect maintenance of septic tanks cannot be expected unless the authority closely monitors it. Therefore this method is suitable for small-scale networks where close supervision is possible. Instead of septic tanks, interceptors without stabilisation of sludge may be used. This method also requires frequent maintenance that has to be done by the community or individuals. This system is thus only suitable in areas with a high level of community participation.

Another alternative is communal septic tanks or interceptors discharging into small-bore sewers. The maintenance and sludge removal in communal septic tanks is easier than in the individual systems. However common lands are required for construction of septic tanks.

The selection from the alternatives is determined by local conditions, in particular with regard to the level of community participation and operation and maintenance.

4.2.2 Primary treatment

Anaerobic ponds and improved communal septic tanks are financially attractive options for the removal of settable organic matter.

Anaerobic ponds are preferred in areas where land price is low. The investment and operational costs of anaerobic ponds are comparatively low and operation is simple. High removal of total BOD can be expected under tropical conditions. Odour nuisance is an important disadvantage.

Improved settling tank with inflow at the bottom of the tank, similar as for UASB is also considered as an appropriate option in areas where land price is comparatively high.

The selection from the alternatives is determined by local conditions in particular with regard to the land price and operation and maintenance.

4.2.3 Post treatment

Low-cost options to be considered are: waste stabilisation ponds, constructed wetlands, trickling filters, macrophytic ponds, fish ponds, rotating biological contactors and land treatment.

Waste stabilisation ponds

Waste stabilisation is an appropriate method for wastewater treatment and reuse for agriculture in areas where land prices are low, climate is warm and operation and maintenance skills are poor. The effluent from waste stabilisation ponds can easily be made suitable for agriculture and high nutrient value in the effluent is an added advantage as fertiliser.

Constructed wetlands

Free Water Surface (FWS) and Subsurface Flow (SF) type wetlands are the two types to be considered. The major problem associated with the FWS type is mosquito breeding. In case of the SF type the water level is kept below the soil surface and mosquito-breeding problem is avoided. The effluent can also easily be made for agriculture. Therefore the Subsurface Flow type constructed wetlands is selected for secondary evaluation.

Sand filtration systems

Sand filtration systems have been successfully used for wastewater treatment in France, Denmark and Morocco as described in Chapter 3.2.3. Since river sand is locally available sand filtration is also considered as an appropriate option for wastewater treatment.

Trickling filters
Trickling filters are comparatively cheap to build and simple to operate, but it is not efficient in removal of pathogenic organisms. Therefore it is appropriate only when agricultural reuse is not applied. Reliability of power supply and availability of skilled personal for operation and maintenance are two important factors affecting the performance of trickling filters which therefore cannot be applied in rural areas. For example clogging of filters due to excessive bio film formation may occur.

Macrophytic ponds
Duckweed has shown good performance with regards to the choice of a macrophytic plant. There are many advantages of using duckweed ponds for wastewater treatment as described in Chapter 3.2.3. The BOD and pathogen removals are also very satisfactory. However there are two conflicting opinions about mosquito control. While one argues that duckweed pond enhances mosquito population the other says it prevents mosquito breeding (Chapter 3.2.3). The disease transmission through mosquitoes (encephalitis, filariasis, malaria and dengue) is one of major health problems in North Western Province. Therefore further research is needed to include this option for wastewater treatment in Sri Lanka. Duckweed technology can be applied in areas where mosquito breeding does not occur (for example: up country).

Fishponds
Fishponds are an attractive method for wastewater treatment in developing countries as it provides employment opportunities as well as nutritious food for the people. However in Sri Lanka promotion of animal farming is rejected due to religious reasons (as the majority are Buddhists). Heavy social resistance against freshwater fish cultivation project is a recent example in this regard. Fishponds can be applied among fishing communities with different religious background. The population densities of the fishing communities in North Western Province are relatively low and therefore on-site sanitation can be provided. Therefore this option is not considered for further evaluation.

Rotating biological contactor treatment systems (RBC)
Rotating bio disks for wastewater treatment are more complicated to build and operate than trickling filters. Removal of pathogenic organisms is also not up to the standard of agricultural reuse. The operation and maintenance problems are generally high. For example keeping the moving items like main shaft, bearings, motors, driving belts in good order is complicated. Hence skilled operators are needed. Reliable power supply is a prime requirement to prevent die-off of the biomass. Relatively high investment cost and frequent breakages of the equipment (main shaft etc.) due to excessive weight of biomass are two disadvantages making RBC unsuitable for local conditions.

Activated sludge treatment systems
The towns in the study are relatively small (less than 100,000 inhabitants). Therefore activated sludge need not to be considered because, capital and operational costs are extremely high. The removal of pathogenic organisms is also far below the standard of agricultural reuse.

Land treatment systems
Direct application of wastewater to a vegetated land (restricted irrigation) is the simplest method of wastewater treatment and reuse. One of the restrictions could be the requirement to provide storage during the rainy season, which is needed to prevent possible pollution of

drinking water sources in the surrounding villages when untreated wastewater is carried away with the surface runoff. Therefore the need for storage must be investigated when land treatment is selected.

5. STRATEGIES FOR SANITATION AND REUSE OF WASTEWATER

The aim of formulating a strategy is to identify sustainable technical solutions to prevent possible groundwater pollution and to reuse wastewater in Sri Lanka where sanitation is not considered under the current urban development programmes. The study area is limited to North Western Province. These strategies developed are applicable in other areas in Sri Lanka, which have the same characteristics (topography, population density etc) as the study area.

The three major towns in North Western Province (population greater than 30,000) namely Kurunega, Puttalam and Chilaw are shown in Figure 5.1. However the provincial capital (Kurunegala) is not considered in this study, as it is included in the urban development programme launched by the Urban Development Authority of Sri Lanka. The other two towns are located in the dry zone and the case study is done for Puttalam town considering agricultural reuse. The housing schemes close to the provincial capital (urban housing schemes) are considered as a separate category. The small townships, villages and smaller housing schemes are the third category considered.

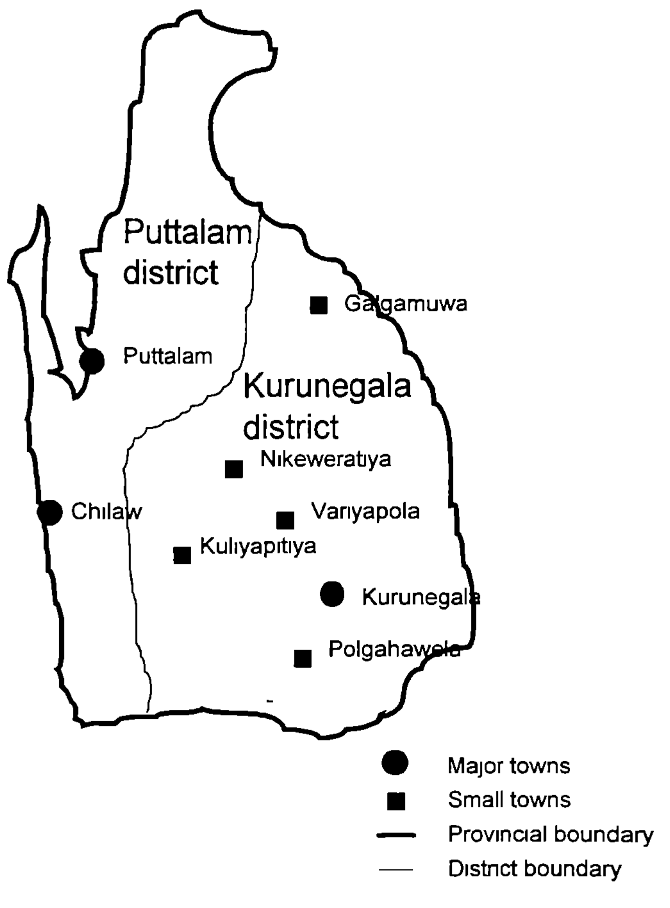

Figure 5.1 Map of the North Western Province (area under study)

5.1 A strategy for major towns

The proposed strategy is aimed to identify appropriate methods to prevent groundwater pollution and to find possibilities to reuse of wastewater in Puttalam, a typical major town in the dry zone.

The existing population in the town is 37,000 capita while the average population density is 80 capita/ha. The population density in the town centre is around 175 capita/ha. The expected growth rate is 1.5% (see chapter 2.2). Then the expected population in the year 2030 is 57,800 capita.

Two collection networks are proposed for the town. The schematic layout of the sewer mains is shown in Figure 5.2. One system collects wastewater from the area close to the sea (Area 2) which cannot be used for agricultural purposes because the wastewater will be saline. The other system collects wastewater from the remaining area and will be used for agricultural purposes.

The water distribution network in the town was commissioned in 1996. Pipes have been laid considering future expansions of the town. Therefore the end points of the wastewater collection network are also selected on the same basis.

5.1.1 Proposal for Area-1

It is assumed that the percentage of connected population in area 1 is 90 % in year 2030. Then the expected population in this area in the year 2030 is around 47,000 capita. The respective surface area is 425 ha.

(a) Wastewater collection and treatment

Wastewater collection

A small-bore sewer system is one of the options to be considered. Disposal of troublesome solids like polythene, cloths etc. has been reported from other sewerage schemes in the island. The wastewater production can also be lower than expected. Therefore pilot studies should be carried out before selecting small-bore sewers as a suitable option.

As described in Chapter 4.2.1, small-bore sewers with individual septic tanks or interceptors are not selected due to lack of close supervision and a low level of community participation expected from employed people in major towns.

Individual people own the lands in major towns. Therefore finding common lands to construct communal tanks and to lay house connections is not practical Therefore small-bore sewers with common septic tanks are not considered as an appropriate option.

Hence the conventional (separate) sanitary sewer system is preferred for Puttalam town. The design procedure and the respective results are presented in Appendix 5.1. The summary of the design results is presented in Table 5.1.

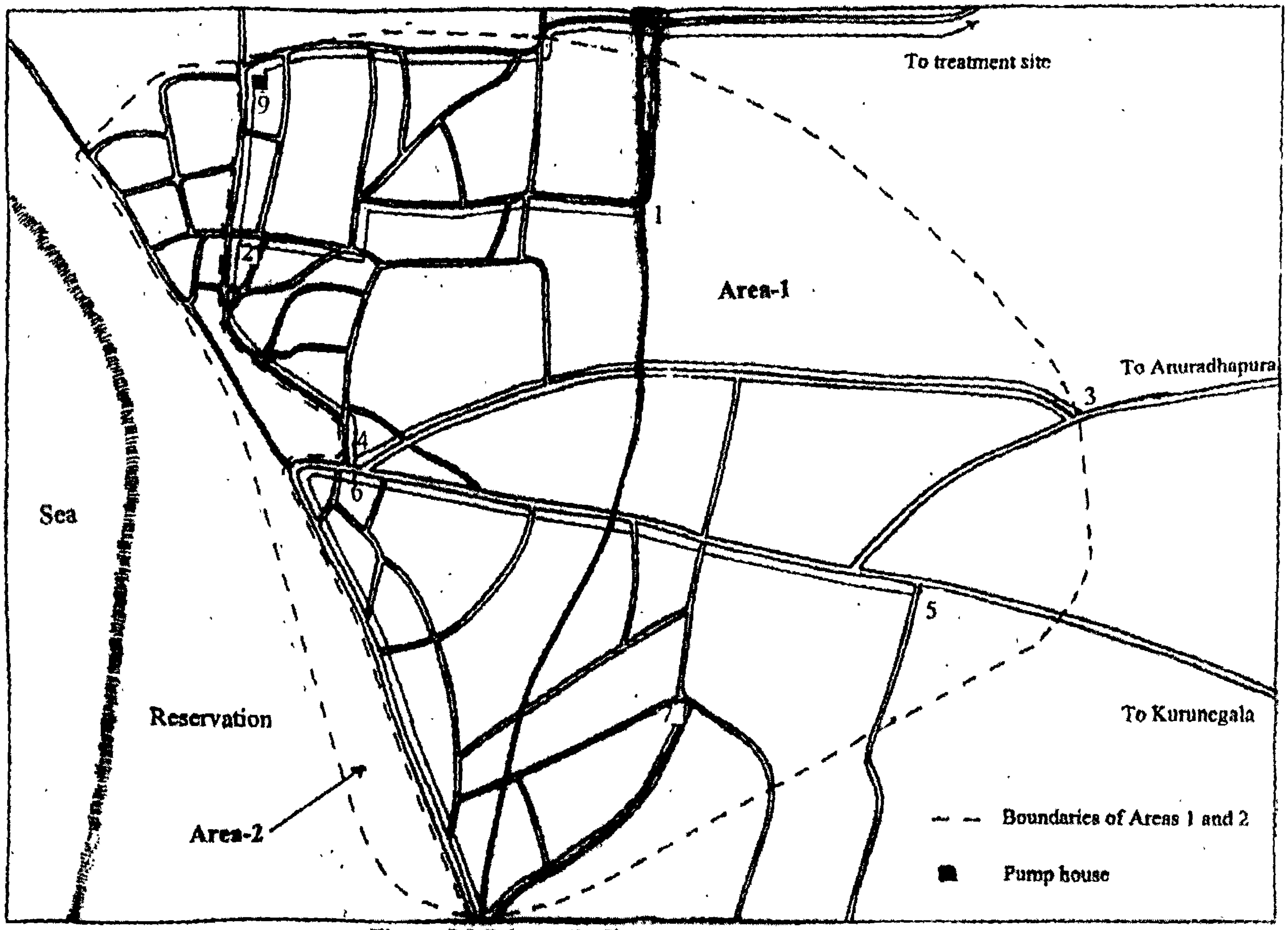

Figure 5.2 Schematic diagram of Puttalam town

Table 5.1 Summary of the design results – collection network

Section	Diameter mm	Length m
1-2	300	1400
3-4	300	2700
5-6	300	2100
7-6	250	2600
6-9	500	1600
Pumping main	300	2900
Secondary sewers	160	69,000

Primary treatment

As described in Chapter 4.2, anaerobic ponds are selected for primary treatment due to very low land cost. The design population is taken as 50,000 capita. The preliminary design procedures and respective results are presented in Appendix 5.1. The summary of the design results is presented in Table 5.2.

Table 5.2 Design results of anaerobic ponds

Parameter	Unit	Value
Design population	capita	50,000
Daily wastewater production	m^3/d	4,050
Influent BOD_5	mg/l	430
No of ponds in parallel	Nos.	2
Surface area per pond (at mid depth)	m^2	1,480
Depth of the pond	m	4
Free board	m	0.5
Desludging frequency	years	2
Effluent BOD_5	mg/l	170

Post treatment

As described in Chapter 4.2 the appropriate technological options suitable for wastewater treatment are: waste stabilisation ponds, constructed wetlands, sand filters, trickling filters and land treatment. However trickling filters and land treatment process are not selected for the wastewater treatment in Puttalam town, as the proposed crops (rice, vegetables, fruits etc) require higher effluent standards.

Therefore waste stabilisation ponds, constructed wetlands and sand filters are considered for evaluation. The design population is taken as 50,000 capita. The preliminary design procedures and respective results are presented in Appendix 5.1. The summaries of the design results are presented in Tables 5.3, 5.4 and 5.5.

Table 5.3 Design results of algae ponds

Parameter	Unit	Value		
		FP1	FP2	MP
General data				
Design population	capita	50,000	50,000	50,000
Daily wastewater production	m^3/d	4,050	4,050	4,050
Influent characteristics				
Influent BOD_5	mg/l	170	70	-
Influent coliform count	FC/100 ml	2.5×10^7	5.8×10^5	2.8×10^4
Pond dimensions				
Volume of a pond	m^3	10,230	9,180	12,480
Surface area per pond (at mid depth)	m^2	6,050	5,740	8,400
No of ponds in parallel	Nos	2	2	2
Average depth	m	1.7	1.6	1.5
Free board	m	0.5	0.5	0.5
Effluent characteristics				
Effluent BOD_5	mg/l	70	30	-
Effluent coliform count	FC/100 ml	5.8×10^5	2.8×10^4	1,000

Table 5.4 Design results of constructed wetlands

Parameter	unit	Value
General data		
Design population	capita	50,000
Daily wastewater production	m^3/d	4,050
Influent characteristics		
Influent BOD_5	mg/l	170
Influent coliform count	FC/100 ml	2.5×10^7
Constructed wetlands		
Surface area per bed	m^2	1,975
Average depth of a bed	m	0.6
Number of beds (in parallel)	Nos.	10
Effluent characteristics		
Effluent BOD_5	mg/l	30
Effluent coliform count	FC/100 ml	1,000

Table 5.5 Design results of sand filters

Parameter	Unit	Value
General data		
Design population	capita	50,000
Daily wastewater production	m^3/d	4,050
Influent characteristics		
Influent BOD_5	mg/l	170
Influent coliform count	FC/100 ml	2.5×10^7
Dimensions of the filter beds		
Surface area per bed	m^2	1,550
Average depth of a bed	m	2
No of beds	Nos.	12
Effluent characteristics		
Effluent BOD_5	mg/l	30
Effluent coliform count	FC/100 ml	1,000

(b) Cost comparisons

The technological alternatives considered for cost comparisons are:
Alternative 1: Collection network + anaerobic ponds + constructed wetlands
Alternative 2: Collection network + anaerobic ponds + sand filters
Alternative 3: Collection network + anaerobic ponds + facultative and maturation ponds

Cost estimates were prepared based on the current rates obtained from NWS&DB and Department of Irrigation in Sri Lanka. The respective cost estimates for different alternatives are presented in Appendix 5.3. The summary of the cost calculations is presented in Table 5.6.

Table 5.6 Cost comparison for different technological options

	Capital cost US$	Annuity US$	Running cost/year US$	Total annual cost US$
Alternative 1	1,705,522	114,769	90,689	205,458
Alternative 2	2,009,543	135,228	86,788	222,016
Alternative 3	1,700,712	114,446	87,879	202,325

(c) Selection of the appropriate alternative

Alternatives 1 and 3 are very similar, as the difference is only 1.5%. Alternative 2 (sand filter) is 10% more expensive.

Since the cost difference between alternatives 1 and 3 is small a technical comparison is made for final evaluation. Waste stabilisation ponds are preferred over constructed wetlands for a major town due to following reasons

- The technology of the waste stabilisation ponds is familiar to the local contractors as small scale irrigation structures exist in the region
- Since many wetland beds are involved, influent distribution of the constructed wetlands is more complex.
- There is no experience of local aquatic plants to be used in the wetland beds

5.1.2 Proposal for Area-2

The expected population in Area 2 by year 2030 is around 6,500 capita. The respective square area is 53 ha. Off-site sanitation is proposed only for the commercial area. The expected population in the commercial area is about 2,600 capita (40%). The expected connected population is around 2,300 capita (90%). The respective surface area is about 12 ha. On-site sanitation is proposed for remaining area.

(a) Wastewater collection and treatment

The wastewater collected form this area will be directed to a post treatment site near the sea. The post treatment includes anaerobic ponds and natural sand beds near the sea The pond effluent will percolate through the sand beds and finally discharged into the ground. The respective design calculations are presented in Appendix 5.2 The summary of the design results is presented in Table 5.7.

Table 5.7 Summary of the design results - wastewater collection and treatment

Description	Unit	value
General		
Population	capita	2,300
Daily wastewater production	m^3/d	185
BOD_5 of raw wastewater	mg/l	435
Collection network		
160mm dia. pipes	m	2,560
Number of manholes	Nos	95
Anaerobic ponds		
Volume	m^3	360
Hydraulic retention time	days	2
Depth	m	2.5
Surface area	m^2	145
Effluent BOD_5	mg/l	175
Number of ponds (parallel)	Nos.	2
Sand filters		
Influent BOD_5	mg/l	175
Influent coliform count	FC/100 ml	2.5×10^7
Total surface area	m^2	880
Depth of the bed	m	2
Effluent BOD_5	mg/l	30
Effluent coliform count	FC/100 ml	1,000

(b) Cost estimates

Cost estimates were prepared based on the current rates obtained from NWS&DB and Department of Irrigation in Sri Lanka The respective cost estimates are presented in Appendix 5.3. The summary of the costs is as below.

Capital cost = US$ 23,676
Running cost = US$ 298

5.1.3 Cost estimate for entire system

The cost estimates for entire system is presented in Table 5.8.

Table 5.8 Summary of costs for wastewater collection and treatment

Area	Capital cost US$	Running cost US$
Area-1	1,700,712	87,879
Area-2	23,676	298
Total	1,724,388	88,177

5.1.4 Agricultural reuse

Two alternative methods proposed for agricultural reuse are as follows.

- Fruit and coconut cultivation in the unutilised lands
- Paddy, vegetables and cereal crop cultivation in the paddy fields and traditional farmlands

Alternative 1: Fruit and coconut cultivation

Fruit and coconut cultivation in the unutilised government lands is focused in this option. The types of fruits proposed are banana, pineapple and papaya. Private sector participation to be obtained by giving these lands to the private sector by a lease contract. The crop production is calculated for a period of 30 years. The respective calculations are presented in Appendix 5.4. The income generated by the crop is presented in Table 5.9.

Table 5.9 Income generated by coconut and fruit cultivation

Period	2000 - 2010	2010 - 2020	2020 - 2030
Annual income (US$)	72,351	104,999	121,322

Alternative 2: Paddy, vegetables and cereal crop cultivation
The aim of this proposal is to supply water for cultivation of paddy, vegetables and cereal crops in the paddy fields and traditional farmlands in the area. The crop production is calculated for a period of 30 years. The respective calculations are presented in Appendix 5.4. The incomes generated by the crops are presented in Table 5.10.

Table 5.10 Income generated by paddy, vegetables and cereals

Period	2000 - 2010	2010 - 2020	2020 - 2030
Annual income (US$)			
Paddy	20,672	23,684	27,366
Cereal and vegetables	24,117	27,631	31,927
Total	44,789	51,315	59,293

5.1.5 Cost recovery
The capital and running costs of agriculture scheme and sewerage scheme are analysed separately. The respective calculations are presented in Appendix 5.5.

(a) Agriculture scheme
The capital cost of the agricultural scheme includes costs of pumping station, pumping main, maturation ponds and distribution of treated effluent up to the agricultural site. The running costs include pumping cost (electricity cost) and 25% of operation and maintenance cost of the whole scheme, including sewerage but excluding electricity.

The life span of each option was considered as 30 years. The real interest rate of 5.3% was used for calculation of the annual costs. The respective annual costs are presented in Table 5.11.

Table 5.11 Annual cost for agriculture scheme

Capital cost US$	Annuity US$	Running cost US$	Annual cost US$
128,000	8,600	27,000	35,600

Annuity factor = 0.067293

Cost recovery from Alternative 1 (coconut and fruit cultivation)
The annual cost (US$ 35,600) is 36% of the annual income (US$99,600) generated by Alternative 1 (Appendix 5 4). The cost that can be charged (from private sector) is assumed as 15%. Therefore cost recovery from alternative 1 is not feasible.

Cost recovery from Alternative 2 (paddy and cereal crop cultivation)
In Sri Lanka water supplied to farmlands from irrigation schemes is not billed. Therefore cost recovery cannot be expected from Alternative 2. However the income generated through paddy, cereal crops and vegetables is attractive for farmers to eliminate their poverty. The government as practised in irrigation schemes should provide the capital and running costs.

(b) Sewerage scheme

The capital cost of sewerage scheme comprises of: costs of collection network, anaerobic ponds and facultative ponds with respect to Area 1 and total estimated cost with respect to Area 2. The running cost includes 75% of operation and maintenance cost of whole scheme excluding electricity.

The life span of each option was considered as 30 years. The real interest rate of 5.3% was used for calculation of the annual costs. The respective annual costs are presented in Table 5.12.

Table 5.12 Annual cost for sewerage scheme

Capital cost US$	Annuity US$	Running cost US$	Annual cost US$
1,596,400	107,500	61,200	168,700

Annuity factor = 0.067293

The annual cost per household is US$ 22.60 (assuming an average connected population of 41,000 capita). An average annual income around US$ 650 per household can be estimated for Puttalam town. Then the annual sewerage cost per household is around 3.5% of the respective annual income and therefore not within affordable limit. Government grant around 75% of capital cost is required to make the proposal feasible.

The tariff structure can be prepared based on the average water consumption as all the water connections are metered. However the low-income community (stand post users) should be billed on a nominal value.

5.1.6 Implementation

The implementation of the sewerage project can be done through NWS&DB as they are the national authority responsible for sewerage and are capable of maintaining the sewerage projects of this calibre.

5.2 A strategy for new urban housing schemes

Provision of infrastructure facilities at the construction stage of the housing schemes is the more effective way of planning. As described in chapter 2.4 the housing schemes formed by private land developers does not consist of basic sanitation facilities. This strategy is to find out the possibilities of providing a sewer network and a treatment system for a new housing scheme at the construction stage.

The basic data like land area per house, land price, number of houses in the housing schemes and household size etc. are considered similar to those of the existing housing schemes. (The basic data of some existing housing schemes are presented in Appendix 2.1). It is also assumed that the treated effluent is to be discharged to the nearby paddy fields through an agricultural stream. The technological options considered and design parameters used are described below.

A typical arrangement of a housing scheme is shown in Figure 5.3 and the respective basic data are presented in Table 5.13.

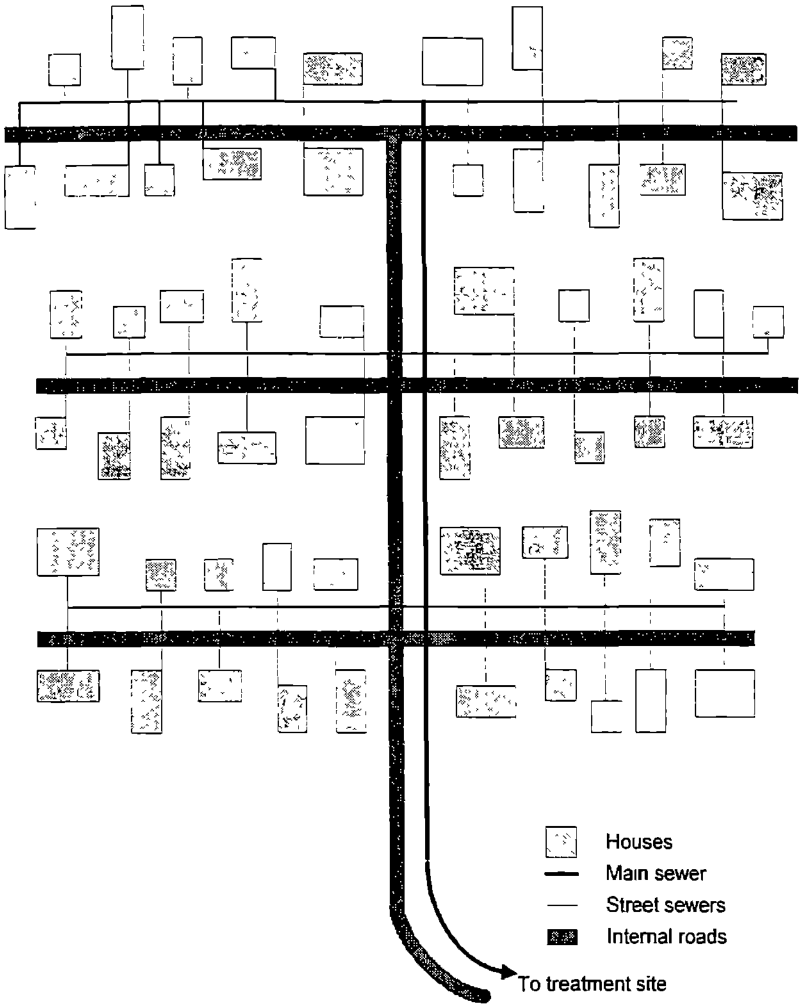

Figure 5.3 Schematic diagram of a typical housing scheme

Table 5.13 Basic data of a typical housing scheme

Description	Unit	Value
Total no of houses	Nos.	100
Household size		5.5
Population	capita	550
Area of a block	ha	0.03
Average land price within the housing scheme/ha	Rs	16,000,000
Population density	Capita/ha	160
Per capita waste production	g/c/d	35
Per capita water consumption	l/c/d	135
Per capita waste water production	l/c/d	108
Influent BOD_5 concentration	mg/l	325
Total length of house connections	m	1,500
Average length of primary sewers	m	1,300
Length of the main sewer	m	1,700
Average land price at secondary treatment site/ha	Rs	6,000,000

5.2.1 Preliminary design of collection network and treatment system

The residents in these housing schemes are of high or middle-income categories and community participation for maintenance of pipelines etc. cannot be expected. The proposed system should be with less maintenance. Pipe laying through common lands is not considered as the residents of this category prefer privacy. Therefore individual and communal septic tanks are not selected. Small-bore sewers are also not considered due the similar reasons as explained in Chapter 5.1.1 (major towns). Hence conventional sewers are considered for these housing schemes. The design criteria used are presented in Appendix 5.6. The summary of the design results is presented in Table 5.14.

Table 5.14 Design results for collection network

Pipe	Diameter (mm)	Length (m)
House connections	110	1,500
Primary sewers in the sub roads	160	1,300
Sewer main up to the treatment site	160	1,700

Primary treatment:

Technological options considered are:

- UASB type septic tank (improved septic tank)
- Anaerobic pond

The design criteria used are presented in Appendix 5.6. The summaries of the design results are presented in Table 5.15.

Table 5.15 Design results for primary treatment

Parameter	Unit	value
UASB type septic tank		
Flow	m^3/d	59
Influent BOD_5	mg/l	324
Desludging frequency	years	0.5
Volume	m^3	20.3
Hydraulic retention time	hours	8.2
Effluent BOD_5	mg/l	130
Anaerobic pond		
Flow	m^3/d	59
Influent BOD_5	mg/l	324
Depth	m	3
Volume of the pond (circular pond)	m^3	149
Hydraulic retention time	days	2.5
Effluent BOD_5	mg/l	130
Desludging frequency	years	2

Post treatment

Since the treated effluent is discharged to the paddy fields trickling filters are not considered for evaluation. In addition to this, maintenance of trickling filters by a small community will be difficult (as described in Chapter 4). Therefore constructed wetlands, waste stabilisation ponds and sand filters are selected for further evaluation. The summary of the design results is presented in Table 5.16.

Table 5.16 Design results for post treatment

Parameter	Unit	value
Constructed wetlands		
Flow	m^3/d	59
Influent BOD_5	mg/l	130
Effluent BOD_5	mg/l	30
Length of the bed	m	12
Width of the bed	m	11
Depth of the bed	m	0.6
No. of beds		2
Sand filters		
Flow	m^3/d	59
Influent BOD_5	mg/l	130
Effluent BOD_5	mg/l	30
Length of the bed	m	11
Width of the bed	m	10
Depth of the bed	m	2
No. of beds		2
Waste stabilisation ponds		
Flow	m^3/d	59
Influent BOD_5	mg/l	130
Effluent BOD_5	mg/l	30
Surface area of the first pond	m^2	180
Average depth of the first pond	m	1.7
Surface area of the second pond	m^2	192
Average depth of the second pond	m	1.6
Total hydraulic time	days	10

5.2.2 Cost estimation and comparison

Cost estimation of different options is done in this section. The life span of each option was considered as 30 years. The real interest rate of 5.3% was used for calculation of the annual costs. Cost estimates were prepared based on the current rates obtained from NWS&DB and Department of Irrigation in Sri Lanka. The respective cost estimates are presented in Appendix 5.7.

Capital cost

The summary of capital costs is presented in Table 5.17.

Table 5.17 Summary of capital costs

Item	Capital cost US$
Collection network	33,219
Primary treatment	
UASB type septic tank	2,942
Anaerobic pond	8,989
Post treatment	
Constructed wetlands	31,407
Sand filters	33,032
Waste stabilisation ponds	34,615

Cost comparison

Cost comparison between primary and post treatment options is done in this step. These cost comparisons are presented in Table 5.18 and 5.19 respectively.

Table 5.18 Cost comparison between primary treatment options

Option	Capital cost US$	*Running cost/year US$
UASB type septic tank	2,942	33
Anaerobic pond	8,989	33

*Cost of desludging

Table 5.19 Cost comparison between post treatment options

Option	Capital cost US$	*Running cost/year US$
Constructed wetlands	31,407	2,134
Sand filters	33,032	2,148
Waste stabilisation ponds	34,615	2,134

*Including staff and machinery for collection network

5.2.3 Technology selection

Primary treatment

The capital cost of the UASB type septic tank is less than that of the anaerobic pond. Land price seems to be the major deciding factor of the capital cost. The disadvantage of the septic tank system is comparatively short desludging frequency (6 months). But vacuum trucks can be hired from the Municipal council and a caretaker system is proposed for maintenance. Therefore this is not a major problem in regular maintenance. Hence the UASB type septic tank is selected as the appropriate option.

Post treatment

The lowest capital cost is for constructed wetlands. The running costs of constructed wetlands and sand filters are almost same. Since this is a very small-scale plant the influent distribution in constructed wetlands is not complicated as that in a large-scale system. However lack of knowledge about local aquatic plants is another disadvantage. But it is expected that this problem can be overcome by doing further research and pilot scale studies. Therefore by considering these facts constructed wetland system is selected as most appropriate

5.2.4 Affordability

Capital cost

The capital cost of the lowest option is US$ 67,568. When this capital cost is included in the land price the respective contribution is US$ 676 per 0.03 ha (12 perch) plot. Then the new average land price per 0.03 ha plot is US$7,288. This is only 10% increase when compared with the original land prices of US$ 6,612 per 0.03 ha plot.

The space requirements of a typical housing unit at present include the house, septic tank and soakage pit, shallow well and some space for gardening. But this space requirement can be reduced considerably by providing offsite sanitation facilities.

Therefore the minimum plot area may be reduced from 0.03 ha to 0.025 ha (10 perches still keeping enough space for a shallow well and gardening. Then the average price per plot is US$ 6,073 and which is even lower than the original land price for a 0.03 ha plot and therefore affordable for the buyers.

Since the developed land with more infrastructure facilities will be more attractive amongst the buyers the land development agencies will be benefited too by the proposed strategy. It will also improve the sanitation in the housing schemes and prevent possible groundwater pollution. Provision of water supply facilities may also be studied in this regard.

Cost recovery
The running cost per household per year is US$ 22. The residents in these housing schemes belong to upper middle class and the average annual income is around US$ 1650. Then the average annual sewerage bill is only 1.3% of the total income and therefore affordable.

5.2.5 Strategy for Implementation

The execution of the construction works should be initiated by the land development agencies at the stage of land development. Government should introduce new regulations regarding land development in order to ensure proper implementation by the respective land development agencies. The necessary technical guidance can be given through the National Water Supply and Drainage Board (NWS&DB) at a reasonable cost.

The routine maintenance is to be carried out by the housing society. A preventive maintenance schedule should be followed in this regard. A caretaker is to be appointed to perform this task. Contract labour can be obtained whenever necessary. The sludge removal to be carried out by a vacuum truck and the assistance of the municipal council should be given to perform this task. The machinery maintenance to be carried out either through the NWS&DB or through a reputed private institution. The necessary technical guidance to be provided by the NWS&DB. The expenses related with all maintenance activities should be bared by the housing society.

A three-tier maintenance policy (governmental policy) to be developed to link the three institutions: housing society, municipal council and NWS&DB as being in practise in the tube well maintenance. This ensures the responsibility of each institution to achieve the common goal.

5.3 A strategy for existing urban housing schemes

The existing housing schemes situated just outside the old town limits will be incorporated to the town development plan as the proposed town limits cover these areas. Therefore only the housing schemes situated about 3–6 km away from the proposed town limits are considered under this study.

Onsite sanitation facilities with septic tanks are available in these housing schemes. But these septic tanks are too close to the drinking water sources (shallow wells) and sometimes not properly sealed. Therefore there is a big risk of groundwater pollution.

The technical options considered to improve the situation are similar as those of the new housing schemes.

5.3.1 Preliminary design and cost estimation

The design procedure is same as that of the new housing schemes. About 30% increase of the capital cost can be expected, as the sanitation system is not planned at the initial stage. Then the capital cost of the system is US$ 87,838. The respective annual cost (including capital and running costs) per household is US$ 81 (annuity factor is 0.067,293). This is about 4.9% of the annual income of a household and therefore too high.

5.3.2 Affordability

The main problem associated with the existing housing schemes is recovery of the capital cost Significant contribution for the capital cost cannot be expected from the community unless it is recovered through a tariff structure. Only 10% recovery of the capital cost is affordable to the community. Therefore government grant up to 90% of the capital cost is required to make the proposal feasible. Then the respective annual cost per household is only 27 US$. This about 1.7% of the annual income of a household and therefore affordable.

5.3.3 Implementation

As a short term measure the existing septic tank systems to be improved (improvement of septic tank systems are discussed in Chapter 5.4). However construction of offsite system in future to be considered. The construction works and operation and maintenance activities are similar to the new housing schemes.

5.4 A strategy for small towns, villages and rural housing schemes

5.4.1 Areas with population density between 50 – 150 capita/ha.

Small towns, rural housing schemes and villages with population density higher than 50 capita/ha belong to this category. On-site sanitation is the existing sanitation method in use. As described in the chapter 2.4 the major problems in the existing sanitation systems are:

- No standard is followed when locating the cesspools and septic tanks
- Construction quality of the cesspools and septic tanks are poor

(a) Proposed system

The soil permeability in the area is moderate (15 – 25 $l/m^2.d$). The groundwater table generally lies below 2m. Since the population density is low provision of off-site systems is expensive. Therefore the existing sanitation systems in use (on-site sanitation) with some improvements are proposed. As described in the chapter 4.1, pour flush toilet is suitable for the area as most people can afford it. A water closet with septic tank is optional for the high-income group.

Locating the septic tanks

The septic tanks and soakage pits should be located a reasonable distance away from the drinking water sources. A distance of 15m will be safe enough. The required standard can be achieved in many cases. Therefore relocating of the septic tanks is an important task in the proposed improvements.

Wastewater disposal

In the existing system the grey wastewater produced from the bathrooms and kitchen is directly discharged on the ground and only the black wastewater is sent to the septic tank. The same method with some improvements can be adopted. The grey water can be discharged to a trench located near a big tree (like coconut tree) as its pathogen content is very low and the BOD removal is important. The black water to be directed to the septic tank and the effluent to be discharged through a soakage pit filled with stones. The effluent discharge through a sand bed in the soakage pit will be safer when the distance between the pit and the drinking water source is less than the required standard.

Improvements in the septic tanks
Improving the quality of the septic tanks is one of the important things to be done. The existing septic tanks should be rehabilitated up to the proposed standard.

The septic tank can be constructed with bricks, as the technology is locally popular. A screed concrete to be provided in order to avoid failures due to uneven settlements. The inside of the tank should be plastered with cement and sand to prevent leakage. The septic tank may be single or double compartment depending on the affordability. The preliminary design of the septic tank is presented in Appendix 5.8.

(b) Cost estimates
The capital costs for single and double compartment septic tanks are US$ 40 and US$ 62 respectively. The respective cost estimates are presented in appendix 5.8. Reduction of 20% - 30% costs may be expected by introducing community participation.

(c) Domestic reuse
The grey wastewater can be treated and reused for toilet flushing and gardening. This becomes important in the houses where individual piped water supply systems are available and shortage of water in the source (during the drought period) exists.

The average grey water production in a typical house is about 300 – 400 l/d and is quite sufficient for reuse purposes. An anaerobic compartment to remove oil and grease as well as certain portion of BOD and sand filter bed to remove remaining BOD and pathogens may be an appropriate method for treatment. However more experimental studies may be necessary to evaluate the situation.

(d) Implementation
As a short-term measure, critical areas where groundwater pollution is higher should be identified and priority should be given for upgrading. Long term planning is necessary to upgrade the entire system. Introduction of an educational programme about preventing the groundwater pollution should also be included for the planning process, as attitude changes are very important for the success.

A standard to be maintained when the new septic tanks are constructed. The local authority together with their monitoring programme for building construction can do the monitoring.

A loan scheme may be necessary for implementation of the proposal. The existing loan scheme for housing development can be extended for this purpose. A government grant (or foreign aid) may also be necessary for the low-income people.

5.4.2 Areas with population density below 50 capita/ha.

(a) Proposed system
The remote areas with low population density (below 50 capita/ha) belong to this category. As described in the chapter 2.2, there are some people in these areas without any sanitation systems in use. As discussed under chapter 4.1, VIP latrine and pour-flush latrine can be introduced for these areas. But most people will use VIP latrine due to its low cost.

(b) Implementation

Community participation is a very important activity in the implementation process. The toilet can be constructed using the local materials. However the slab and the ventilation pipe should be distributed free of charge among low-income people in order to obtain successful results.

6 CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

1. Off-site sanitation in new urban housing schemes is technically and financially feasible. The capital cost of the sanitation system should be included in the land price.

2. An appropriate treatment of wastewater from urban housing schemes includes: improved septic tank (UASB type) as primary treatment and constructed wetlands as post treatment. The treated effluent is suitable for paddy cultivation. The land price seems to be the deciding factor for selecting the treatment method.

3. The housing society should do maintenance and recovery of running cost of the sewerage scheme. A caretaker system is proposed for routine maintenance. The assistance of the municipal council and NWS&DB is proposed for sludge removal and major repairs.

4. The same sanitation system can be introduced for existing urban housing schemes. However recovery of only 10% of the capital cost is possible through a sewerage tariff. Therefore a government grant up to 90% of the capital cost is required for implementation. The strategy for maintenance is similar as that of new urban housing schemes.

5. As a short-term proposal, the sanitation system in existing urban housing schemes (on-site sanitation) should be rehabilitated and maintained with close supervision of the authority.

6. Off-site sanitation with agricultural reuse in Puttalam town is feasible only when the government grant up to 90% of the capital cost (including capital cost of the agricultural scheme) and running cost of the agricultural scheme are available. Waste stabilisation ponds are the appropriate treatment method.

7. The running cost and 11% of the capital cost of the sewerage scheme can be recovered by a sewerage tariff.

8. The reuse of wastewater can either be used in traditional farmlands or in uncultivated land (with private sector participation). The income generated from traditional farmlands helps to eliminate poverty of farmers in the area. However cost recovery is not feasible through both alternatives.

9. On-site sanitation is proposed for small towns, rural housing schemes and villages. Pour-flush toilet and water closet with septic tank (for high-income group) are the appropriate sanitation systems for areas of population density above 50 capita/ha. Pour-flush and VIP latrines are the appropriate sanitation systems for remote areas with a population density below 50 capita/ha.

10. A standard to be maintained when new septic tanks are constructed. The local authority together with their monitoring programme for building construction can do the monitoring. The septic tank may be single or double compartment depending on the affordability. The construction cost can be reduced by 20 – 30% with community participation.

11. The existing loan scheme for housing development can be extended to provide financial assistance for construction and rehabilitation of toilets. A government grant (or foreign aid) may also be necessary for the low-income people.

6.2 Recommendations

It is recommended that:

1. Off-site sanitation be provided for urban housing schemes at the construction stage of a scheme.

2. The application of alternative treatment options for housing schemes be investigated, with particular attention to constructed wetlands, by establishing and monitoring pilot plants and studying performances of BOD and pathogen removal under different conditions (aquatic plants, filter media, loading rates etc.).

3. An appropriate method be investigated for provision of piped water facilities for urban housing schemes.

4. The application of shallow and small-bore sewer network systems be investigated, by establishing such systems in urban areas and assessing technical performances and involvement of community participation in maintenance activities.

5. Quality of drinking water sources be monitored regularly and remedial actions be taken to rehabilitate poor sanitation systems in use.

6. A legal framework and regulations for on-site sanitation be promulgated, and standard specifications be developed for locating and constructing septic tanks.

7. Experimental studies be carried out to investigate possibilities of grey water reuse and application of sludge as a fertiliser.

8. Social surveys be conducted for the planing and designing of new technological options for sanitation.

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APPENDICES

APPENDIX 2

Water consumption pattern

Table A.2.1 Water consumption pattern of different regions in Sri Lanka

Region	Water connections (Nos.)			Consumption (m^3/month)			PCC
	Dom.	Com.	Total	Dom.	Com.	Total	(l/c/d)
Greater Colombo	177,946	14,226	192,172	4,563,989	1,047,907	5,611,896	177
Kurunegala	25,941	2,268	28,209	583,687	45,621	629,308	135
Kandy	32,229	1,626	33,855	563,305	28,643	591,948	106
Anuradhapura	12,781	1,777	14,558	253,692	33,799	287,491	120
Bandarawela	12,865	1,217	14,082	215,153	21,113	236,266	102
Kalutara	22,401	849	23,250	356,484	21,856	378,340	100
Ratnapura	15,289	1,702	16,991	300,495	35,924	336,419	120
Hambanthota	15,781	936	16,717	315,691	29,002	344,693	125
Matara	29,740	1,277	31,017	715,588	32,963	748,551	146
Trincomalee	8,387	169	8,556	173,144	6,582	179,726	127

Note: 1. Dom. – Domestic 2. Com. – Commercial 3. PCC – Per capita consumption
Source. NWS&DB

Table A.2.2 Water consumption pattern of stand posts in different regions

Region	Number of stand posts	Per capita consumption (estimated) l/c/d
Greater Colombo	4530	30
Kurunegala	164	38
Kandy	689	25
Anuradhapura	193	40
Bandarawela	1002	20
Kalutara	201	28
Ratnapura	578	30
Hambanthota	1409	40
Matara	688	37
Trincomalee	407	35

Source: NWS&DB

Table A.2.3 Population served and water consumption in North Western Province

Water supply scheme	Population served capita	Total population capita	% served	Per capita water consumption (l/c/d)
Puttalam	32,200	37,000	87	138
Chilaw	21,000	32,000	67	137
Dankotuwa	2,100	5,100	41	134
Nattandiya	2,750	5,800	47	130
Wennappuwa	1,200	3,500	34	130
Variyapola	2,,850	7,900	36	133
Nikeweratiya	4,600	6,600	70	129
Galgamuwa	4,200	7,800	54	135
Gokerella	2,100	4,600	46	131
Dodangaslanda	900	2,300	39	125
Ridigama	1,200	2,800	43	128
Girıulla	2,300	5,500	42	132
Pannala	1,650	4,450	37	135

Source: NWS&DB – North Western Region

Water quality

Table A.2.4 Bacteriological quality of water collected from shallow wells

Location	Date	Total coliform per 100ml	Faecal coliform per 100ml
Tumbulla	30/12/98	600	320
Tumbulla	30/12/98	620	400
Kudarawaliya	30/12/98	>1000	>1000
Divullepitiya	30/12/98	680	550
Divullepitiya	30/12/98	>1000	>1000
Timbiriyawa	15/02/99	465	268
Hulawa	15/02/99	507	428
Hulawa	15/02/99	715	460
Halmillawa	08/05/99	500	265

Source: NWS&DB – North Western Region

Table A.2.5 Chemical quality of water collected from shallow wells

Location	Turb NTU	PH	Cond µs/cm	Hard mg/l	N-NH4 mg/l	N-NO^{2-} mg/l	N-NO^{3-} mg/l	Cl^- mg/l	PO^{4-} mg/l	Fl mg/l	Fe^{3+} mg/l
Pannala town-Negombo road	8.5	6.9	234	168	0.06	0.02	8.8	104	0.34	0.26	0.15
Pannala town-K'pitiya Rd	18	7.3	673	190	0.23	0.01	4.84	236	0.34	0.42	0.17
Kuliyapitiya town	1.6	7	296	236	0.05	0.01	12.3	44	0 53	0.41	0.11
Puttalam town-coast	2.2	7.9	1194	508	0.48	0.02	60.3	276	4.72	0.36	0.26
Nakkawatte village	23	6.8	230	216	0.23	0.02	3.96	24	0 4	0.11	0.04
Nikeweratiya -Magalle lake	2.5	7.8	472	344	0.23	0.05	70.4	44	1.42	1.73	0.13
Pannala town-K'gala Rd	20	7.7	697	270	0.31	0.01	7.48	192	0.33	0.29	0.49
Slimline factory-Pannala	2.4	7.6	125	148	0.03	0.01	14.5	28	0.22	0.16	0.05
Nikeweratiya - site office	1.6	7.5	548	300	0.1	0 01	3.52	84	0 59	1.42	0.02
Magulagama	2.1	7.5	731	660	0.92	0.05	83.2	564	0.97	0.83	0 12
Variyapola town-Chilaw road	1.7	7 3	625	245	0.25	0 02	45	52	0.95	0.41	0.14
Wannigama-village	1.8	7.5	525	288	0.09	0.01	5 7	78	0.52	1.2	0.02
Hettipola town-Chilaw road	1.6	7 2	325	265	0.06	0.01	18.5	62	0.65	0.5	0.12
Bihalpola Village	16	6.9	260	245	0.26	0.01	5.75	35	0.4	0.19	0.13
Variyapola town-K'gamu Rd	1.7	7.6	550	340	0.15	0.01	8.5	56	0.64	0.52	0.21
Meegahakotuwa village	1.6	7.2	275	225	0.24	0.02	5.75	36	0.64	0.45	0.11

Source: NWS&DB – North Western Region

Information about urban housing schemes

Table A.2.6 Land utilisation and prices of four existing housing schemes

Location	No. of land blocks	Area of a land block perches	Land price per 1 perch Rs
Malkaduwawa	112	15 - 20	25,000 - 45,000
Wehera	76	10 - 15	50,000 - 65,000
Heeressagala	68	8 - 15	45,000 - 65,000
Ampitiya	118	8 - 15	40,000 – 60,000

Source: District Land Registry

APPENDIX 5.1 – DESIGN CALCULATIONS FOR AREA 1

1 WASTEWATER COLLECTION

(i) Preliminary design of the collection system

Minimum slopes for ordinary sewers

The tractive force (τ in N/m^2) required for grit transport is given by:

$$\tau = \rho_w gRS$$

Where
ρ_w = Density of water in kg/m^3
R = Hydraulic radius in m
S = Sewer gradient

It is assumed that the required tractive force is 1.2 N/m^2. Using this value the minimum slope of sewers flowing can be calculated. The respective results are presented in Table A.5.1.1 (The schematic diagram for the sewer mains is presented in Chapter 5.1).

Table A.5.1.1 Minimum slopes required for the ordinary sewers

Diameter (mm)	Minimum slope (1:1000)
160	3.1
200	2.5
250	2.0
300	1.6
400	1.2
450	1.1
500	1.0

Sewer diameters

The ground slope in the town is generally greater than 3.5$^0/_{00}$. Therefore the pipe slope is following the ground slope. The charts prepared based on the flow formula Prandtl – Colebrook / Colebrook – White were used for evaluation of the sewer diameters. The peak factor used is 1.8. The minimum diameter of the house connections and primary sewers is 160mm. The respective results for the main sewers are presented in Table A.5.1.2.

Table A.5.1.2 Diameter of the main sewers in the collection network

Section	Area of contribution ha	Q l/s	Qpeak l/s	Qfull l/s	Diameter mm	Length m
1-2	121.3	13 36	24.05	53.44	300	1,400
3-4	126.8	13.97	25.14	55.86	300	2,700
5-6	105.1	11 58	20.84	46.3	300	2,100
7-6	72.4	7.97	14.35	31.9	250	2,600
6-9	425.6	46.88	84.38	187.5	500	1,600
					Total length	**10,400**

Primary sewers
The density of the distribution pipes in the existing water distribution network is 133 m/ha. Since 90% of houses to be connected to the sewer collection network the density of the primary sewers is higher than that of the water distribution network. Expecting 20% increase the density of primary sewers is considered as 160 m/ha. Then the respective length of the primary sewers in Area 1 (425.6 ha) is 68,096.

Manholes in the sewers
Assuming that the manholes to be provided in 30m intervals and additional 10% to be provided in the junctions and other important places,

No. of manholes in the main sewers	=	1.1*(10,300/30)
	=	377 Nos.
No. of manholes in the secondary sewers	=	1.1*(68,096/30)
	=	2,500 Nos.

(ii) Preliminary design of the Pumping mains
The wastewater collected to be pumped to an agricultural area outside the town boundary. A pumping main is proposed to transport the raw wastewater up to the waste stabilisation ponds and then to discharge by gravity to the agricultural area. The respective calculations are presented in Table A.5.1.3.

Table A.5.1.3 Calculations for raw wastewater pumping mains

	Diameter mm	Flow l/s	Velocity m/s	Length m	Friction loss m	Static head m	Total head m
Pumping main	300	56.3	0.8	2900	9.6	11	20.6

2 WASTEWATER TREATMENT

(i) Primary treatment by anaerobic ponds

The BOD load to the anaerobic pond is given by;

$$L_v = \frac{Q*C_i}{1000}$$

Where L_v = BOD load to the anaerobic pond (kg BOD_5/d)
Q = Wastewater inflow (m^3/d)
C_i = Influent BOD_5 concentration (mg/l)

Then the volumetric loading rate can be calculated from the formula;

$$\lambda_v = \frac{L_v}{V} = \frac{Q*C_i}{V*1000}$$

Where λ_v = Volumetric loading rate (kg BOD/m^3.d)
V = Volume of the pond (m^3)

Since V = A*d, the above equation can be rearranged as;

$$A = \frac{Q*C_i}{\lambda_v*d*1000}$$

Where A = Pond area (m^2)
d = Average depth of the pond (m)

Since the air temperature is higher than 20 ^{0}C the λ_v value can be taken as 0.3 kg BOD_5/m^3.d (Veenstra *et al.*, 1998). Then the above equation can be rearranged as;

$$A = \frac{Q*C_i}{300*d}$$

From the above equation pond dimensions are calculated. Two ponds of same dimensions to be used in order to ensure the safe performance even during the desludging period. The length to width ratio of 2 – 3 to 1 is preferred to avoid sludge banks forming near the inlet (Mara., 1992).

Operation of the anaerobic ponds
By having the two anaerobic ponds (including the stand by pond) in parallel operation following advantages can be obtained.

- Capable to absorb peak loads
- Ensures better effluent quality and prevents arising of anaerobic conditions in the facultative ponds
- Bottom seal (clay layer) is protected.
- Settled solids distributed over more surface area

However provision should be made to isolate one anaerobic pond during the desludging period.

Desludging of the anaerobic ponds
The two ponds are operated in parallel until one pond becomes 50% full. Then the respective pond is emptied for desludging while other pond is in operation. The desludging operation of the second pond is carried out after that of the first pond The desludging period for a pond is assumed as 6 months.

Effluent BOD_5 concentration
Since the minimum air temperature is above 20^{0}C, 60% BOD removal can be expected from an anaerobic pond. Therefore the effluent BOD_5 concentration from the anaerobic pond C_e is given by;

$$C_e = 0.4* C_i$$

Where C_e = Effluent BOD_5 concentration (mg/l)

The respective design results are presented in Table A.5.1.4

Table A.5.1.4 Design parameters and results for the anaerobic pond

Parameter	Unit	Value
General data		
Designed population	capita	50,000
Per capita wastewater production	l/c/d	81
Influent characteristics		
Flow	m^3/d	4,050
Influent BOD_5	mg/l	435
Pond dimensions		
No of ponds in parallel	Nos.	2
Surface area required	m^2	1,468
Designed area (mid depth)	m^2	1,482
Length at mid depth	m	39
Width at mid depth	m	38
Top length of the pond	m	49
Top width of the pond	m	48
Bottom length of the pond	m	31
Bottom width of the pond	m	30
Depth of the pond	m	4
Free board	m	0.5
Slope in the embankment		1:2
Effluent characteristics		
Desludging frequency	years	1.9
% BOD removal		60%
Effluent BOD_5	mg/l	174

(ii) Secondary treatment by constructed wetlands

Considering the subsurface horizontal flow wetland systems the saturated cross sectional area for the flow through the system can be calculated according to Darcy's law. Then the velocity of the flow is given by;

$$\frac{Q}{A_c} = k_s * S$$

Where
A_c = Cross sectional area of the wetland bed perpendicular to the flow direction (m^2)
Q = Average flow rate through the system (m^3/d)
k_s = Hydraulic conductivity (permeability) of the medium ($m^3/m^2.d$)
S = Slope of the wetland bed

Then the above equation can be rearranged as

$$A_c = w*d = \frac{Q}{k_s * S}$$

Where
w = Width of the wetland bed (m)
d = Depth of the wetland bed (m)

The river sand with coarse grains is available in the North Western Province, which can be used as the media of the constructed wetland bed. The k_s value for the coarse sand is 480 $m^3/m^2/d$ (table 3.15 of the literature study). The aquatic plant type selected is *Phragmites australis* and the maximum root penetration of this plant is about 600mm (table 3.16 of the literature study). Therefore the bed depth is selected as 600mm.

Substituting these values the above equation can be rearranged to calculate the width as:

$$w = Q/2.88$$

Length of the wetland bed

For plug flow conditions

$$\frac{C_e}{C_i} = e^{-K_T \times t}$$

Where
- C_i = Influent BOD_5 concentration (mg/l)
- C_e = Effluent BOD_5 concentration (mg/l)
- K_T = Rate constant at the temperature T ^0C (d^{-1})
- t = Hydraulic retention time (days)

After integration the above equation can be rearranged as;

$$K_T * t = \ln C_i - \ln C_e$$

$$t = \frac{\ln C_i - \ln C_e}{K_T}$$

The hydraulic retention time "t" is given by the following formula;

$$t = \frac{V*p}{Q} = \frac{(l*w*d)*p}{Q}$$

Where
- V = Volume of the bed (m3)
- d = Depth of submergence (m)
- p = Porosity of the bed as a fraction
- l = Length of the wetland bed (m)

Combining these two formulas;

$$l = \frac{Q*(\ln C_i - \ln C_e)}{K_T*d*p*w}$$

The rate constant K_T is given by the equation;

$$K_T = K_{20}*(1.1)^{(T-20)}$$

Where
- K_{20} = Temperature dependant first order rate constant at 20 ^0C (d^{-1})
- T = Temperature ^0C

For the coarse sand K_{20} value can be taken as 1.35 (d^{-1})

Hence $K_T = 1.35*(1.1)^{(23-20)}$
$= 1.8$ (d^{-1})

For coarse sand the porosity (p) is 0.35.
The effluent BOD_5 standard (C_e) is 30 mg/l

Substituting these values in the previous equation;

$$l = Q*(\ln C_i - \ln 30)/(1.797*0.6*0.35*w)$$

Substituting the value "w" the above equation can be rearranged as;

$$l = 7.63*(\ln C_i - \ln 30)$$

Using the above equation for the different inflow BOD_5 concentrations the respective lengths can be calculated. The respective design results are presented in Table A.5.1.5.

Table A.5.1.5 Design results for constructed wetlands

Parameter	Unit	Value
General data		
Designed population	capita	50,000
Per capita wastewater production	l/c/d	81
Influent characteristics		
Flow	m^3/d	4,050
Influent BOD_5	mg/l	174
Dimensions of the wetland beds		
Length of the bed	m	14
Width of the bed	m	141
Depth of the bed	m	0.6
Surface area of a bed	m^2	1,975
No of beds	Nos	10
Aquatic weed		Phragmites australis
Effluent characteristics		
% BOD removal		83 %
Effluent BOD_5	mg/l	30

(iii) Secondary treatment by algae ponds

Design of facultative ponds

First facultative pond in the series
Assuming completely mixed conditions in the first facultative pond, the formula for BOD removal is given by:

$$\frac{C_i}{C_{1e}} = (1+K_T*t_o)$$

Where C_i = Influent BOD_5 concentration to the facultative ponds (mg/l)

C_{1e} = Effluent BOD_5 concentration from the first pond (mg/l)
t_o = Hydraulic retention time (days)

The degradation coefficient K_T is given by;

$$K_T = K_{20}*(1.056)^{(T-20)}$$

Where K_T = Temperature dependant coefficient at T ^{0}C (d^{-1})
K_{20} = Degradation coefficient at 20 ^{0}C = 0.25 (d^{-1})
T = Temperature of the wastewater in ^{0}C

Assuming the mean temperature at the coldest month is 23 ^{0}C

$$K_T = 0.25*1.056^{(23-20)}$$
$$= 0.294\ (d^{-1})$$

To prevent anaerobic conditions arise in the facultative ponds the BOD concentration in the first facultative pond should be kept below 70 mg/l. Assuming completely mixed conditions, the effluent concentration of the first facultative pond should be limited to 70 mg/l too. Then the above equation can be rearranged as:

$$t_o = \frac{1}{20.58}*(C_1\text{-}70)$$

At the same time the hydraulic retention time "t_o" can be calculated from the relationship;

$$t_o = V_1/Q$$

Where Q = Wastewater flow (m^3/d)
V_1 = Volume of the first facultative pond (m^3)

Combining the above two equations;

$$V_1 = \frac{Q}{20.58}*(C_1\text{-}70)$$

From the above equation the volume of the first facultative pond and hence the pond dimensions can be calculated. Assuming an average depth of 1.7m the surface area of the first facultative pond can be calculated.

A number of facultative ponds may be used to obtain the required effluent standard. This is in particular important for the reduction of faecal coliforms. The minimum detention time is determined by the life cycle of algae.

Assume "n" facultative ponds in series (after the first facultative pond) are used for BOD removal under completely mixed conditions. Then the BOD removal by the facultative ponds is given by the equation;

$$\frac{C_{21}}{C_e} = \left(1+K_T*\frac{t}{n}\right)^n$$

Where
C_{2i} = Influent BOD_5 concentration to the second pond (mg/l)
C_e = Effluent BOD_5 concentration from the last pond (mg/l)
t = Total hydraulic retention time of the ponds in series (days)
n = No of ponds in series after the first pond
K_T = 0.294 (d^{-1})

The hydraulic retention of a single pond t is thus;

$$\frac{t}{n} = \frac{1}{K_T} * \left[\left(\frac{C_{2i}}{C_e} \right)^{\frac{1}{n}} -1 \right]$$

The volume of a single pond in series "V" is given by:

v = Q*t/n

Substituting the influent BOD_5 concentration of 70 mg/l and the required effluent standard (effluent concentration of the last pond) of 30 mg/l and combining the above two equations

$$v = \frac{Q}{0.294} * \left[\left(\frac{70}{30} \right)^{\frac{1}{n}} -1 \right]$$

From the above equation the dimensions of the facultative ponds in series (after the first facultative pond) can be calculated. The respective design results are presented in Table A.5.1.6.

Table A.5.1.6 Design results for facultative ponds

Parameter	Unit	Value	
		FP1	FP2
General data			
Design population	Capita	50,000	50,000
Per capita wastewater production	l/c/d	81	81
Influent characteristics			
Total wastewater flow	m^3/d	4,050	4,050
Influent BOD_5	mg/l	170	70
Influent coliform count	FC/100 ml	2.5×10^7	5.8×10^5
Pond dimensions			
No. of ponds in parallel	Nos.	2	2
Volume of one pond	m^3	10,230	9,180
Surface area required per pond	m^2	6,020	5,739
Designed area per pond	m^2	6,050	5,740
Length at mid depth	m	121	164
Width at mid depth	m	50	35
Top length	m	129	172
Top width	m	58	43
Bottom length	m	116	159
Bottom width	m	45	30
Depth	m	1.7	1.6
Free board	m	0.5	0.5
Slope		1.3	1.3
Effluent characteristics			
% BOD removal		60	57
Effluent BOD_5	mg/l	70	30
Effluent coliform count	FC/100 ml	5.8×10^5	2.8×10^4

Faecal coliform removal by the facultative ponds

Faecal coliform removal by the first facultative pond

Assuming completely mixed conditions, the faecal coliform removal by the first facultative pond is given by the equation;

$$\frac{S_i}{S_{1e}} = (1+k_b*t_o)$$

Where S_i = No of faecal coliforms in the influent (FC numbers/100ml)
S_{1e} = No of faecal coliforms in the effluent (FC numbers/100ml)
t_o = Hydraulic retention time (days)

The die-off coefficient k_b is temperature dependent and is given by the equation;

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

Where k_b = Die – off rate coefficient (day^{-1})
T = Minimum sewage temperature at the coldest month in ^0C

Assuming $T = 23^0C$,

$$k_b = 2.6*(1.19)^{(23-20)}$$
$$= 4.38 \text{ day}^{-1}$$

Assume that the faecal coliform count in the influent of the first facultative pond S_i is 10^8 (FC/100 ml).

Substituting this value in the previous equation, the effluent faecal coliform count from the first facultative pond S_{1e} is expressed by;

$$S_{1e} = \frac{10^8}{[1 + 4.38*t_o]}$$

Assuming completely mixed conditions, the faecal coliform removal by the ponds in series is given by the equation;

$$\frac{S_{2i}}{S_e} = \left(1 + k_b * \frac{t}{n}\right)^n$$

Where	S_{2i}	= No of faecal coliforms to the second pond (FC/100ml)
	S_e	= No of faecal coliforms leaving the last pond (FC/100ml)
	n	= No of ponds in series (nos.)
	t	= Total hydraulic retention of ponds in series (days)

Since the effluent faecal coliform count from the first facultative pond is equal to the influent coliform count to the second facultative pond ($S_{1e} = S_{2i}$), the effluent coliform count from the last facultative pond s_e is given by:

$$S_e = \frac{S_{1e}}{\left[1 + 4.38 * \frac{t}{n}\right]^n}$$

Using these equations, for different pond arrangements and hydraulic retention times the respective effluent faecal coliform count from the facultative ponds can be calculated.

Faecal coliform removal by the maturation ponds

The calculation procedure is same as that of the maturation ponds in series. The volumes of the maturation ponds are calculated using the respective hydraulic retention times calculated using the above formulas. The respective design results are presented in Table A.5.1.7.

Table A.5.1.7 Design results for maturation ponds

Parameter	Unit	Value
Influent characteristics		
Wastewater flow	m^3/d	4050
Influent faecal coliform count	FC/100ml	2.8×10^4
Pond dimensions		
No of ponds in parallel	Nos.	2
Volume of a pond	m^3	12,480
Surface area required per pond	m^2	8,320
Designed surface area per pond	m^2	8,400
Length at mid depth	m	168
Width at mid depth	m	50
Top length	m	176
Top width	m	58
Bottom length	m	163
Bottom width	m	45
Depth	m	1 5
Free board	m	0 5
Slope of the embankments		1:3
Effluent Characteristics		
Effluent faecal coliform count	FC/100ml	1,000

(iv) Post treatment by sand filtration

Since most the biological actions in the sand filters takes place closer to the surface of the filter bed the surface loading rate is selected as the design criteria. The results are checked for the average hydraulic loading rates during the flooding period.

The BOD load to the sand filter bed is given by;

$$L_v = \frac{Q*C_i}{(1000)}$$

Where
- L_v = BOD load to the sand filter bed (Kg of BOD_5/d)
- Q = Wastewater inflow (m^3/d)
- C_i = Influent BOD_5 concentration (mg/l)

Then the surface organic loading rate SOL can be expressed as;

$$SOL = \frac{L_v}{A_s} = \frac{Q*C_i}{(1000*A_s)}$$

Where
- SOL = Surface organic loading rate (Kg of $BOD_5/m^2.d$)
- As = Surface area of the filter bed (m^2)

The above equation can be rearranged as;

$$A_s = \frac{Q*C_i}{(1000*SOL)}$$

- A_s = Surface area of the filter bed (m^2)
- d = Depth of the sand bed (m)

Since Sri Lanka is a warm country the loading rates applied at Ben Sergo, Morocco is safe enough for the design. Therefore the respective surface organic loading rate (SOL) is taken as 76 g. of COD/m^2.d. The depth of the sand bed is assumed as 2m (similar to that of Morocco) too.

Assuming the typical COD/BOD ratio as 2 the OSLR for Sri Lankan conditions is considered as 0.38 kg. BOD$_5$/m^2.d. Using this value the above equation can be simplified as;

$$A_s = \frac{Q*C_i}{38}$$

Using the above equation the dimensions of the filter beds can be calculated.

Hydraulic loading

The surface hydraulic loading rate (for a full cycle) used in the treatment plant at Ben Sergo is 0.14 m^3/m^2.d. Assuming the same pattern of wastewater loading to be applied in the proposed treatment plant (3 days drying period and 2 days flooding period) the maximum surface hydraulic loading rate is taken as 0.14 m^3/m^2.d.

Then the respective surface hydraulic loading rate (SHL) for a full cycle can be calculated as;

$$SHL = \frac{Q*n_f}{A_s*n_t}$$

Where SHL = Surface hydraulic loading rate (m^3/m^2.d)

n_f = Period of flooding (days)

n_t = Total cycle (days)

The value of SHL should be below the allowable rare of 0.14 m^3/m^2.d. The respective design results are presented in Table A.5.1.8.

Table A.5.1.8 Design results for sand filters

Parameter	Unit	Value
General data		
Designed population	capita	50,000
Per capita wastewater production	l/c/d	81
Influent characteristics		
Wastewater flow	m^3/d	4,050
Influent BOD$_5$	mg/l	170
Dimensions of the filter beds		
Surface area of a bed	m^2	1,550
Length of a bed	m	62
Width of a bed	m	25
Sand depth	m	2
No of beds	Nos.	12
Effluent characteristics		
% BOD removal		83
Effluent BOD$_5$	mg/l	30

APPENDIX 5.2 – DESIGN CALCULATIONS FOR AREA 2

The population in Area 2 by year 2030 is 6,500 capita. Since part of the area is away from the commercial centre onsite sanitation can be introduced for that part of the community. It is assumed that onsite sanitation can be introduced for 60% of the total population in this area. Therefore the population to be served through offsite sanitation is 2,600 capita (40% of the population). The expected connected population is 2,300 capita.

1 Wastewater collection

(i) Preliminary design of the collection system

The following calculations are for the collection network consisting of 420 houses (for an average household size of 5.5). The pipe slope is following the ground slope.

Length of the pipe lines

As calculated in Appendix 5.1 the length of the sewer lines are calculated based on the expected sewer network density of 160 m/ha. Then the total length of the sewer collection network is 1,920 m for a contributing area of 12 ha.

Manholes in the sewers

Assuming that the manholes to be provided in 30m intervals and additional 10% to be provided in the junctions and other important places,

Number of manholes in a single collection network = 1.1*(1,920/30)

= 64 Nos.

2 Wastewater treatment

(i) Primary treatment by anaerobic ponds

Two anaerobic ponds are required for the area consisting of 420 houses. The design calculations are similar to that of in Area 1 as presented in Appendix 5.1. The respective design results are presented in Table A 5.2.1.

Table A.5.2.1 Primary treatment by anaerobic ponds

Description	Unit	value
Population	capita	2,300
Flow	m^3/d	185
Influent BOD_5	mg/l	430
Volume of the pond	m^3	356
HRT	days	2
Depth	m	2.5
Surface area	m^2	145
Length at mid depth	m	12
Width at mid depth	m	12
Slope		1:2
No. of ponds (parallel)		2

(ii) Secondary treatment by sand filtration

The effluent from the anaerobic pond is directed to a sand bed (sand bed formed near the lagoon using the natural sand available) along the lagoon and allowed to percolate to the ground. The surface area of the sand bed is calculated using the same loading rates used in Appendix 5.1.

Then the total surface area required is 1,000 m^2. Four 22 × 10 m sand beds to be formed using natural sand. The respective design results are presented in Table A.5.2.2.

Table A.5.2.2 Design results for secondary treatment by sand beds

Description	Unit	value
Influent BOD_5	mg/l	175
Influent coliform count	FC/100 ml	2.5×10^7
Total surface area	m^2	900
Depth of the bed	m	2
Effluent BOD_5	mg/l	30
Effluent coliform count	FC/100 ml	1,000

APPENDIX 5.3 - COST CALCULATIONS

1 Cost calculations for Area 1

(a) Capital cost

(i) Wastewater collection

The capital cost for the wastewater collection is presented in Table A.5.3.1.

Table A.5.3.1 Capital cost estimate for wastewater collection network

Description	unit	Quantity	Rate (Rs)	Amount (Rs)
Laying of 160mm dia. PVC pipes (type600)	Lm	68,096	355	24,174,080
Laying of 250mm dia. PVC pipes (type600)	Lm	2,600	969	2,519,400
Laying of 300mm dia. PVC pipes (type600)	Lm	6,200	1,300	8,060,000
Laying of 500mm dia. concrete pipes	Lm	1,600	4,500	7,200,000
Laying of 300mm dia. PVC pipes (type600)	Lm	2,900	1,300	3,770,000
Excavation for the structures	cum	9,137	300	2,741,291
Grade 20 concrete in the structures	cum	2,073	5,000	10,364,530
Construction of the pump houses	Allow	1	30,000	30,000
Machinery	Allow	1	2,810,000	2,810,000
Caretaker quarters	Allow	2	350,000	700,000
Excavation of the pipelines	cum	44,495	330	14,683,522
Overheads 15%				11,557,923
Total				88,610,746

Note: 1 US$ = Rs. 72.60

(ii) Wastewater treatment by anaerobic ponds and constructed wetlands

The respective cost estimate is presented in Table A.5.3.2.

Table A.5.3.2 Cost estimate for anaerobic ponds with constructed wetlands

Cost estimate for anaerobic ponds				
Description	unit	Quantity	Rate (Rs)	Amount
Site clearing	ha	1.38	150,000	207,000
Land area	ha	1.38	125,000	172,500
Excavation of the pond	m^3	7,156	300	2,146,800
Compaction of the dam	m^3	5,790	500	2,895,000
Clay lining	m^2	5,045	71	358,195
R.M lining to protect wave action	m^3	349	229	79,921
1:2:4 concrete in the structures	m^3	15.4	5,000	77,000
Screed concrete	m^3	5.6	3,800	21,280
450mm dia. concrete pipes	m	50	4,500	225,000
315mm dia. PVC pipes	m	108	1,300	140,400
Pipe specials and sluice gates	Item	1	2,5000	25,000
Overheads 15%				952,214
Sub total 1				**7,300,310**
Cost estimate for constructed wetlands				
Description	unit	Quantity	Rate (Rs)	Amount
Land	ha	4.45	125,000	556,250
Site clearing	ha	4.45	150,000	667,500
Excavation for the structures	m^3	2,430	300	729,000
Excavation of the pipe lines	m^3	339	330	111,870
Compaction of the soil	m^3	2,613	500	1,306,500
200 mm thick clay layer	m^2	4,544	71	322,624
Filter media	m^3	13,618	530	7,217,540
Rock medi at the inlet and outlet	m^3	1,945	620	1,205,900
Screed concrete in the structures	m^3	420	3,800	1,596,000
1:2.4 concrete in the structures	m^3	470	5,000	2,350,000
Rubble masonry work	m^3	2,810	900	2,529,000
Plastering of the structures	m^2	9,262	88	815,056
450mm dia concrete pipes	m	226	7,000	1,582,000
160mm dia PVC pipes	m	1,440	580	835,200
O&M Equipment - Bush cutters	Item	3	30,000	90,000
O&M Equipment – T W tractor	Item	1	150,000	150,000
O&M Equipment - Pick up	Item	1	1,200,000	1,200,000
O&M Equipment - Sludge pumps	Item	1	30,000	30,000
Planting aquatic weed	Item	1	975,000	975,000
Overheads 15%				3,640,416
Sub total 2				**27,909,856**
Capital cost for the combination				**35,210,166**

Note. 1 US$ = Rs. 72.60

(iii) Wastewater treatment by anaerobic ponds and sand filters

The respective cost estimate is presented in Table A.5.3.3.

Table A.5.3.3 Cost estimate for anaerobic ponds with sand filters

Cost estimate for anaerobic ponds				
Description	unit	Quantity	Rate (Rs)	Amount (Rs)
Site clearing	ha	1.38	150,000	207,000
Land cost	ha	1.38	125,000	172,500
Excavation of the pond	m^3	7,156	300	2,146,800
Compaction of the dam	m^3	5,790	500	2,895,000
Clay lining	m^2	5,045	71	358,195
R.M lining to protect wave action	m^3	349	900	314,100
Concrete (1:2.4) in the structures	m^3	15.5	5,000	77,500
Screed concrete in the structures	m^3	5.6	3,800	21,280
450mm dia. concrete pipes	m	50	4,500	225,000
315mm dia. PVC pipes	m	108	1,300	140,400
Pipe specials	Item	1	25,000	25,000
Overheads 15%				987,416
Sub total 1				**7,570,191**
Cost estimate for sand filters				
Description	unit	Qty	Rate (Rs)	Amount (Rs)
Land cost	ha	4.22	125,000	527,500
Site clearing	ha	4.22	150,000	633,000
Excavation for the structures	m^3	16,644	300	4,993,200
200 mm thick clay layer	m^2	18,600	71	1,320,600
Sand media	m^3	37,200	530	19,716,000
Gravel media	m^3	5,775	620	3,580,500
Screed concrete in the structures	m^3	402	3,800	1,527,600
Concrete (1:2:4) in the structures	m^3	569	5,000	2,845,000
Rubble masonry work	m^3	5,305	900	4,774,500
500mm dia. concrete pipes	m	326	4,500	1,467,000
160mm dia. PVC pipes	m	990	355	351,450
110mm dia. PVC pipes	m	7,800	188	1,466,400
Pipe specials	Item	1	25,000	25,000
Overheads 15%				6,484,163
Sub total 2				**49,711,913**
Cost of the combination				**57,282,104**

Note: 1 US$ = Rs. 72.60

(iv) Wastewater treatment by waste stabilisation ponds

The respective cost estimate is presented in Table A.5.3.4.

Table A.5.3.4 Cost estimate for waste stabilisation ponds

Cost estimate for anaerobic ponds				
Description	unit	Quantity	Rate (Rs)	Amount (Rs)
Site clearing	ha	1.38	150,000	207,000
Land cost	ha	1.38	125,000	172,500
Excavation of the pond	m^3	7,156	300	2,146,800
Compaction of the dam	m^3	5,790	500	2,895,000
Clay lining	m^2	5,045	71	358,195
R.M lining to proect wave action	m^3	349	900	314,100
Concrete (1:2:4) in the structures	m^3	15.5	5,000	77,500
Screed concrete in the foundations	m^3	5.6	3,800	21,280
450mm dia. concrete pipes	m	50	4,500	225,000
315mm dia PVC pipes	m	108	1,300	140,400
Pipe specials	Allow	1	25,000	25,000
Overheads 15%				987,416
Sub total 1				**7,570,191**
Cost estimate for facultative and maturation ponds				
Description	unit	Quantity	Rate (Rs)	Amount (Rs)
Site clearing	ha	8.22	150,000	1,233,000
Land	ha	8.22	125,000	1,027,500
Excavation of the pond	m^3	15,984	300	4,795,200
Compaction of the dam	m^3	23,280	500	11,640,000
Clay lining	m^2	34,756	71	2,467,676
R.M lining to proect wave action	m^3	1,800	229	412,200
R:M erosion pad	m^3	9	229	2,061
Concreting in the structures	m^3	15 2	5,000	76,000
Screed concrete in foundation	m^3	5.6	3,800	21,280
500mm dia concrete pipes	m	84	4,500	378,000
315mm dia. concrete pipes	m	164	1,300	213,200
Pipe specials and sluice gates	Item	1	25,000	25,000
O&M Equipment - Bush cutters	Item	2	30,000	60,000
O&M Equipment – T W tractor	Item	1	150,000	150,000
O&M Equipment - Pick up	Item	1	1,200,000	1,200,000
O&M Equipment - Sludge pumps	Item	1	30,000	30,000
Overheads				3,559,668
Sub total 2				**27,290,785**
Cost of the combination				**34,860,976**

Note: 1 US$ = Rs. 72.60

(b) Running cost

(i) Running costs for anaerobic ponds, constructed wetlands and collection network

The respective cost estimate is presented in Table A.5.3.5.

Table A.5.3.5 Running cost

Description	Cost (Rs)/year
Desludging	239,516
Staff	2,604,000
Maintenance	3,261,445
Electricity	479,051
Total	6,584,012

Note: 1 US$ = Rs. 72.60

(ii) Running cost for anaerobic pond, sand filters and collection network

The respective cost estimate is presented in Table A.5.3.6.

Table A.5.3.6 - Running cost

Description	Cost/year
Desludging	239,516
Staff	2,388,000
Maintenance	3,194,245
Electricity	479,051
Sub total	6,300,812

Note: 1 US$ = Rs. 72.60

(iii) Running cost for waste stabilisation ponds and collection network

The respective cost estimate is presented in Table A.5.3.7.

Table A.5.3.7 - Running cost

Description	Cost/year
Desludging	239,516
Staff	2,460,000
Maintenance	3,201,445
Electricity	479,051
Sub total	6,380,012

Note: 1 US$ = Rs. 72.60

(c) Annual cost

The summary of the annual costs for different alternatives is presented in Table A.5.3.8.

Table A.5.3.8 Summary of the annual cost

	Capital cost Rs	Annuity factor	Annuity Rs	Running cost Rs	Total Rs
Alternative 1	123,820,912	0.067293	8,332,257	6,584,012	14,916,268
Alternative 2	145,892,850	0 067293	9,817,539	6,300,812	16,118,351
Alternative 3	123,471,722	0.067293	8,308,759	6,380,012	14,688,770

Note: 1 US$ = Rs. 72.60

2 Cost calculations for Area 2

(a) Capital cost

The capital cost for wastewater collection and treatment is presented in Table A.5.3.9.

Table A.5.3.9 Capital cost for wastewater collection and treatment

Description	unit	Qty	Rate	Amount
Land for anaerobic pond and filters	ha	0.14	50000	7000
Site clearing	ha	0.14	100000	14000
Laying of 160 mm diameter pipes	Lm	1920	355	681600
Excavation of the pond and structures	cum	850	300	255000
Compaction of the embankment	cum	196	500	98000
Clay layer	sqm	407	71	28897
Concreting in the manholes	cum	42	5000	210000
Preparation of two sand beds	Allow	1	200000	200000
15% overheads				224000
Total				1718497

Note 1 US$ = Rs. 72 60

(b) Running cost

The running cost for the wastewater collection and treatment is presented in Table A.5.3.10.

Table A.5.3.10 Running cost for wastewater collection and treatment

Description	Cost (Rs)/year
Desludging	9,600
Maintenance	12,000
Total	21,600

Note 1. Staff cost and machinery maintenance cost included to area 1
Note 2: 1 US$ = Rs. 72.60

3 Cost calculations for total system

The cost calculation for entire area is presented in Table A.5.3.11.

Table A.5.3.11 Cost estimate for entire area

	Capital cost Rs	Running cost Rs
*Area 1	123,471,722	6,380,012
Area 2	1,718,497	21,600
Total	125,190,219	6,401,612

*Waste stabilisation ponds as treatment Note: 1 US$ = Rs. 72.60

APPENDIX 5.4 – AGRICULTURAL REUSE

1 Agricultural reuse in the unutilised government lands – Alternative 1
Treated effluent is used for coconut and fruit crop cultivation in unutilised government lands. The types of fruit proposed are banana, pineapple and Papua. The expected income generated by the respective crops per ha per year is Rs. 150,000 during first ten years and is Rs. 190,000 between 10^{th} and 30^{th} years.

The average application rate for the crop is 5 mm/d. The water available, extents of crops and average income levels are presented in Table A.5.4.1.

Table A.5.4.1 Income generated from the coconut and fruit

Period	2000 - 2010	2010 - 2020	2020 - 2030
Wastewater flow (m^3/d)	2,697	3,084	3,521
*Available for irrigation (m^3/d)	1,751	2,006	2,318
Extent of land (ha)	35	40.1	46.4
Annual income (Rs)	5,252,675	7,622,959	8,807,989

*Assuming evaporation losses of 0.007 $m^3/m^2.d$ and application efficiency of 70%

2 Agricultural reuse in the traditional farmlands – Alternative -2
Treated effluent is used for paddy, vegetables and cereal crop cultivation in the paddy fields and traditional farmlands. The growth period of paddy and cereal crops is 3 months each. Paddy and cereal crops are grown in alternative seasons. The incomes generated from paddy and cereals per ha per year (for 2 growing seasons for each crop) are Rs. 60,000 and Rs. 50,000 receptively.

The average application rates for paddy and cereal are 7 mm/d and 5 mm/d respectively. The water available, extents of crops and average income levels are presented in Table A.5.4.2.

Table A.5.4.2 Income generated from paddy, cereals and vegetables

Period	2000 - 2010	2010 - 2020	2020 - 2030
Wastewater flow (m^3/d)	2,697	3,084	3,521
*Available for irrigation (m^3/d)	1,751	2,006	2,318
Extent of land (ha)			
Paddy	25	28.7	33.1
Cereal and vegetables	35	40.1	46.4
Annual income (Rs)			
Paddy	1,500,764	1,719,464	1,986,764
Cereal and vegetables	1,750,892	2,006,042	2,317,892
Total	3,251,656	3,725,506	4,304,656

*Assuming evaporation losses of 0.007 $m^3/m^2.d$ and application efficiency of 70%

APPENDIX 5.5

Capital and running costs for agriculture and sewerage

The capital and running costs of agricultural scheme and sewerage scheme are analysed separately.

The capital cost of the agricultural scheme includes costs of pumping station, pumping main, maturation ponds and transport of effluent up to the agricultural site. The running costs include all pumping cost (electricity) and 25% of operation and maintenance cost of whole scheme excluding electricity.

The capital cost of sewerage scheme comprises of: costs of collection network, anaerobic ponds and facultative ponds with respect to Area 1; total estimated cost with respect to Area 2. The running cost includes 75% of operation and maintenance cost of whole scheme excluding electricity.

(a) Agricultural scheme

Capital cost
The respective cost estimate is presented in Table A.5.5.1.

Table A.5.5.1 Capital costs of the agricultural scheme

Item	Cost (Rs)
Pumping main	4,143,000
Pumping station	1,386,000
Maturation ponds	3,323,000
Distribution up to agricultural site	420,000
Overheads	23,000
Total	9,295,000

Running cost
The respective running costs are presented in Table A.5.5.2.

Table A.5.5.2 Running costs of the agricultural scheme

Item	Cost (Rs)
Pumping cost (electricity)	479,000
25% of O&M cost (excluding electricity)	1,481,000
Total	1,960,000

(b) Sewerage scheme

Capital cost

The respective cost estimate is presented in Table A.5.5.3.

Table A.5.5.3 Capital costs of the sewerage scheme

Item	Cost (Rs)
Capital cost contribution of Area 1	114,177,000
Total capital cost of Area 2	1,719,000
Total	115,896,000

Running cost

The respective running cost is presented in Table A.5.5.4.

Table A.5.5.4 Running costs of the sewerage scheme

Item	Cost (Rs)
75% of O&M cost (excluding electricity)	4,442,000

APPENDIX 5.6

Design calculations for wastewater collection and treatment in a urban housing scheme

(i) Basic data assumed for the design calculations:

The basic data for a new housing scheme are assumed based on the data available on typical housing schemes. The average per capita water consumption is assumed similar to that of the towns in the North Western province as a typical house in these housing schemes consist of an individual water supply system with a shallow well, a pumping system and an overhead tank. The average per capita wastewater production is assumed as 80% of the average per capita water consumption. The influent BOD_5 concentration is calculated using the average per capita waste production (in terms of BOD_5) of 35 g/c/d. The respective figure was obtained from a housing scheme (Raddoluwa housing scheme) in the adjacent province where sewerage facility is available. It is also assumed that the effluent is transported through the main sewer up to the secondary treatment works situated 1.5 km away from the boundary of the housing scheme. The basic data used for the design calculations are given in Table 5.12 of Chapter 5.2. The schematic diagram of a typical housing scheme is presented in Figure 5.3 of Chapter 5.2.

(ii) Preliminary design of the collection system

Minimum slopes for ordinary sewers

The tractive force (τ in N/m^2) required for grit transport is given by:

$$\tau = \rho_w gRS$$

Where
ρ_w = Density of water in kg/m^3
R = Hydraulic radius in m
S = Sewer gradient

It is assumed that the required tractive force required for conventional sewers is 1.2 N/m^2 and for sewers followed by septic tanks is 0.9 N/m^2. The density of water is 1,000 kg/m^3. Substituting these values in the above equation the required minimum slope for average flow conditions can be calculated. The minimum slopes calculated for different diameters of ordinary sewers are presented in Table A.5 6.1.

Table A.5.6.1 Minimum slopes required for the ordinary sewers

Diameter	Minimum slope for conventional sewers $^0/_{00}$	Minimum slope for sewers after septic tanks $^0/_{00}$
110	4.4	3 3
160	3.1	2 3
200	2.5	1.8
280	1.7	1 3
315	1.5	1 2

Sewer diameters

Generally a ground slope of around 3 - 4 $^{0}/_{00}$ is maintained during land preparation stage in order to discharge storm water through road drains. Therefore it is assumed that the sewers follow the ground slope. The charts prepared based on the flow formula Prandtl – Colebrook / Colebrook – White were used for evaluation of the sewer diameters. The peak factor used is 2.5. The diameter of the house connections is 110 mm. The diameter selected for sewer network (keeping a minimum diameter of 160 mm) is 160 mm.

(iii) Preliminary design of the primary treatment

UASB type communal septic tank

Assuming the sludge production rate of 0.04 m^3/capita/year the volume of the septic tank can be calculated. It is assumed that he desludging frequency is 6 months and the tank to be desludged when 60% full. The respective results are presented in Table A.5.6.2.

Table A.5.6.2 Design results for UASB type septic tank

Design parameter	Unit	Value
Flow	m^3/d	59
Influent BOD_5	mg/l	324
Sludge production rate	m^3/capita/year	0.04
Desludging frequency	years	0.5
Volume required	m^3	18.3
Designed volume	m^3	20.3
Depth of the liquid retention	m	4
Length	m	2.25
Width	m	2.25
Gas collector height	m	1.2
Hydraulic retention time	hours	8.2
Effluent BOD_5	mg/l	130

Anaerobic pond

The design procedure is similar to that of the Puttalam town as described in the appendix-5.3. The respective design results are presented in Table A.5.6.3.

Table A.5.6.3 Design results for anaerobic pond

Parameter	Unit	value
Flow	m^3/d	59
Influent BOD_5	mg/l	324
Volumetric organic loading rate	kg.BOD_5/m^3.d	0.3
Depth	m	3
Volume of the pond (circular pond)	m^3	149
Diameter at top level	m	16
Hydraulic retention time	days	2.5
Effluent BOD_5	mg/l	130
Desludging frequency	years	0.5

(iv) Post treatment

Since the treated effluent is to be discharged to the near by paddy fields the effluent standards should satisfy the WHO guidelines category - GroupB. Therefore waste stabilisation ponds, constructed wetlands and sand filtration are considered as most appropriate options. The design procedures are similar to those of the Puttalam town as described Appendix 5.1. The respective design results are presented in Chapter 5.2.

APPENDIX 5.7

Cost estimates of different technological options in urban housing schemes:

1 Capital cost

Table A.5.7.1 Cost estimate for collection network

Description	unit	Quantity	Rate (Rs)	Amount (Rs)
Excavation	m^3	2,273	330	750,222
Pipe laying - 110mm dia	m	1,500	188	282,000
Pipe laying - 160mm dia.	m	3,000	355	1,065,000
Overheads				314,500
Sub total				2,411,722

Table A.5.7.2 Cost estimate for UASB type septic tank

Description	unit	Quantity	Rate (Rs)	Amount (Rs)
Land	ha	0.005	6,000,000	30,000
Site clearing	ha	0.005	150,000	750
Excavation for the structures	m^3	91.2	300	27,360
Screed concrete	m^3	0.95	3,800	3,610
1:2:4 Concrete in the structures	m^3	12.8	5,000	64,000
Piping	Allow	1	60,000	60,000
Overheads	Allow			27,900
Total				213,620

Table A.5.7.3 Cost estimate for anaerobic pond

Description	unit	Quantity	Rate (Rs)	Amount (Rs)
Land	ha	0.06	6,000,000	360,000
Site clearing	ha	0.06	150,000	9,000
Excavation in the structures	m^3	92	300	27,600
Earth compaction	m^3	279	500	139,500
Clay layer for sealing	m^2	90	71	6,390
Piping etc	Allow	1	25,000	25,000
Overheads				85,124
Total				652,614

Table A.5.7.4 Cost estimate for post treatment by constructed wetlands

Description	unit	Quantity	Rate (Rs)	Amount (Rs)
Land	ha	0.16	6,000,000	960,000
Site clearing	ha	0.16	150,000	24,000
Excavation for the structures	m^3	165	300	49,500
Compaction of the soil	m^3	215	500	107,500
200 mm thick clay layer	m^2	308	71	21,868
Filter media	m^3	176	530	93,280
Rock media at the inlet and outlet	m^3	29	620	17,980
1:3:6 Screed concrete	m^3	27	3,800	102,600
1:2.4 concrete in the structures	m^3	22	5,000	110,000
Rubble masonry work	m^3	131	900	117,900
Plastering in the structures	m^2	433	88	38,104
Pipe laying - 110mm dia PVC pipes	m	22	188	4,136
Pipe specials	Item	1	25,000	25,000
Construction of the pump house	Item	1	30,000	30,000
Installation of the lifting pump	Item	1	100,000	100,000
O&M Equipment	Item	1	60,000	60,000
Planting aquatic weed	Item	1	120,000	120,000
Overheads				298,248
Total				2,280,116

Table A.5.7.5 Cost estimate for post treatment by sand filters

Description	unit	Qty	Rate	Amount
Land	ha	0.16	6,000,000	960,000
Site clearing	ha	0.16	150,000	24,000
Excavation for the structures	m^3	275	300	82,500
200 mm thick clay layer	m^2	220	71	15,620
Supply and packing of the sand media	m^3	440	530	233,200
Gravel media at inlet and outlet	m^3	77.4	620	47,988
Screed concrete	m^3	19.7	3,800	74,860
1:2:4 concrete in the structures	m^3	25.2	5,000	126,000
Rubble masonry work	m^3	260	900	234,000
160mm PVC pipes (underdrain)	m	62	355	22,010
110mm PVC pipes (underdrain/ inlet)	m	240	188	45,120
Pipe fittings	Allow	1	30,000	30,000
Construction of the pump house	Allow	1	30,000	30,000
Installation of the lifting pump	Allow	1	100,000	100,000
Machinery	Nos.	1	60,000	60,000
Overheads				312,795
Total				2,398,093

Table A.5.7.6 Cost estimate for post treatment by waste stabilisation ponds

Description	unit	Quantity	Rate (Rs)	Amount (Rs)
Site clearing	ha	0 17	150,000	24,900
Land	ha	0 17	6,000,000	996,000
Excavation for the structures	m^3	991	300	297,300
1:3:6 Concrete	m^3	31	3,800	117,800
1:2:4 Concrete	m^3	48	5,000	240,000
RM work	m^3	268	900	241,200
Clay lining to protect seepage	m^2	384	71	27,264
315mm dia. pipes(inlet/inter pond)	m	16	1,300	20,800
Pipe specials	Item	1	30,000	30,000
Machinery	Item	1	60,000	60,000
Construction of the pump house	Item	1	30,000	30,000
Supply and installation of the pumps	Item	1	100,000	100,000
Overheads				32,7790
Total				2,513,054

2 Running cost

The running costs of primary treatment and post treatment are presented in Tables A.5.7.7 and A.5.7.8 respectively. The running cost of the collection system has been included to the each post treatment option as same staff and machinery is used for entire system.

Table A.5.7.7 Running costs for primary treatment (desludging cost)

Option	Running cost per year (Rs)
UASB type septic tank	2,400
Anaerobic pond	2,400

Table A.5.7.8 Running costs for post treatment

Option	*Running cost per year (Rs)
Constructed wetlands	154,965
Sand filters	156,005
Waste stabilisation ponds	154,965

*Including running cost of the collection network

APPENDIX 5.8 – ON-SITE SANITATION

1 Design of the septic tanks

Single compartment septic tank

The volume of the septic tank is given by

$$V = P \times n \times s$$

Where V = Volume required for sludge and scum accumulation (m^3)
P = Population served by the tank (capita)
n = Desludging frequency (years)
s = Sludge accumulation rate (m^3/capita/year)

Taking the average household size as 5.5 capita, sludge accumulation rate as 0.04 m3/capita/year, and desludging period as 5 years:

$$V = 5.5*5*0.04 = 1.1\ m^3$$

Therefore keeping a free board of 0.3m the overall tank dimensions become 1.4 × 0.7 × 1.5 (m × m × m).

Double compartment septic tank

The volume of the second compartment is one third of total volume Then the dimension of the septic tank is as follows.

First compartment = 1.4 × 0.7 × 1.5 (m × m × m)
Second compartment = 0.7 × 0.7 × 1.5 (m × m × m)

2 Cost estimates

Cost estimate for single compartment septic tank

The respective estimate is presented in Table A.5.8.1.

Table A.5.8.1 Cost estimate for single compartment septic tank

Description	unit	Quantity	Rate (Rs)	Amount (Rs)
Excavation	m^3	2.55	125	318
Screed concrete	m^3	0.09	3,800	342
4.5 inch brick walls	m^2	7.28	109	794
Plastering the walls	m^2	7.28	88	641
Piping	Allow	1	150	150
Cover slab	m^3	0.08	5,000	400
Other overheads	Allow			265
Total				2,910

Cost estimate for double compartment septic tank

The respective estimate is presented in Table A.5.8.2.

Table A.5.8.2 Cost estimate for double compartment septic tank

Description	unit	Quantity	Rate (Rs)	Amount (Rs)
Excavation	m^3	3.6	125	450
Screed concrete	m^3	0.12	3,800	456
4.5 inch brick walls	m^2	10.43	109	1,137
Plastering the walls	m^2	14.63	88	1,287
Piping	Allow	1	150	150
Cover slab	m^3	0.12	5,000	600
Other overheads	Allow			400
Total				4.480