2. REVIEW OF SELECTED TECHNOLOGIES

2.1. Introduction

This chapter reviews the main treatment unit processes identified and selected for implementation in this study. The different unit processes were selected based on their likely ‘appropriateness’ for practical implementation in the urban and suburban context in Sri Lanka, in keeping with the requirements discussed in section 1.3.1. Technologies were selected based on a review of available literature, current practices in the field, and on personal field experience.

The intention was to treat each technology as an independent unit process, to be combined as necessary according to the specific site constraints and requirements, in order to arrive at the most appropriate treatment system for each field situation. This could either be as individual, on-plot systems, or as semi-collective systems in combination with simplified and settled sewer networks.

Septic tanks were selected because they provide good primary treatment and are often the most appropriate primary treatment unit process for the other technologies under consideration. They are also in widespread use throughout the country, and significant cost savings could be achieved by using existing septic tanks as primary treatment units. They have been considered separately, rather than together with soakage pits and seepage fields, as is commonly the case. However, in order to fully utilize their potential, a more rational approach to their design was sought in place of the commonly used ‘ad-hoc’ approaches.

All the other technologies considered are, essentially, secondary or tertiary treatment processes. Anaerobic filters were considered mainly for secondary treatment. Horizontal flow reed beds, infiltration-percolation beds and vertical flow planted gravel filters were considered as either secondary or tertiary processes depending on the particular application.

2.2. Septic tanks.

2.2.1. Background

Septic tanks have been the preferred form of on-site sewage treatment for many centuries. In the western world, the first reported use of a septic tank was in France around 1860 (Metcalf and Eddy, 1991). In Britain, the first recorded use of a septic tank for sewage treatment was in 1896 at Exeter (Barwise, 1901). Dual chamber septic tanks, with automated siphons for intermittent effluent disposal, were introduced in the United States in 1884 (Cotteral and Norris, 1969). Besides these recorded instances in the west, recent archaeological excavations have revealed the use of septic tanks in one form or another, as far back as the 4^{th} century B.C. in Sri Lanka, and it is likely that other civilizations prevailing at the time in the East would have had similar systems.

At the present time, septic tanks continue to be widely used for on-site sewage treatment throughout the developed world as well as the developing world. In the US, over 17 million housing units (a third of all housing units in the US), depend on septic tanks (Hershaft, 1976) and they continue to be installed in approximately 25 percent of new housing units (US EPA, 1980). In Malaysia, 37 percent of the total population are served by septic tanks, compared to just 5 percent served by sewer systems (Pillay, 1994). In Brazil, more than 100 million people depend on on-site septic tank systems (Philippi et al, 1999). In Sri Lanka, it is the most widely used form of domestic sewage disposal, with over 1.7 million people in Greater Colombo alone (i.e. over 80 percent of the metropolitan population) dependent on on-site systems (Fernando, 1994).

Significantly, while septic tanks are usually used in rural areas of the developed countries, they are widely used in urban areas of developing countries where few cities have central sewer networks and the coverage remains poor for the ones that do.

2.2.2. Physical description

A septic tank typically consists of a single or multiple-compartment buried tank, where scum, grease and settleable solids are removed from the influent wastewater by gravity separation. The settled solids form a sludge layer at the bottom of the tank. Grease and other light materials float to the surface forming a scum layer as they accumulate. The organic matter retained in the tank undergoes facultative and anaerobic decomposition and is converted into more stable compounds and gases such as carbon dioxide, methane and hydrogen sulphide[11]. Even though the settled solids undergo continuous anaerobic digestion, there is always a net accumulation of sludge in the tank. Some of the material from the bottom layer of the tank is buoyed up by the decomposition gases and sticks to the underside of the scum layer, increasing its thickness. This gradual build up of the scum and sludge layers reduces the effective volumetric capacity of the tank, necessitating the contents to be emptied periodically. The frequency of emptying depends on the design of the tank and the loading. The settled and skimmed wastewater flows from the clear space between the sludge and scum layers to a soil absorption field or a secondary treatment unit. Figure 2-1 shows a schematic section of a typical dual-compartment septic tank.

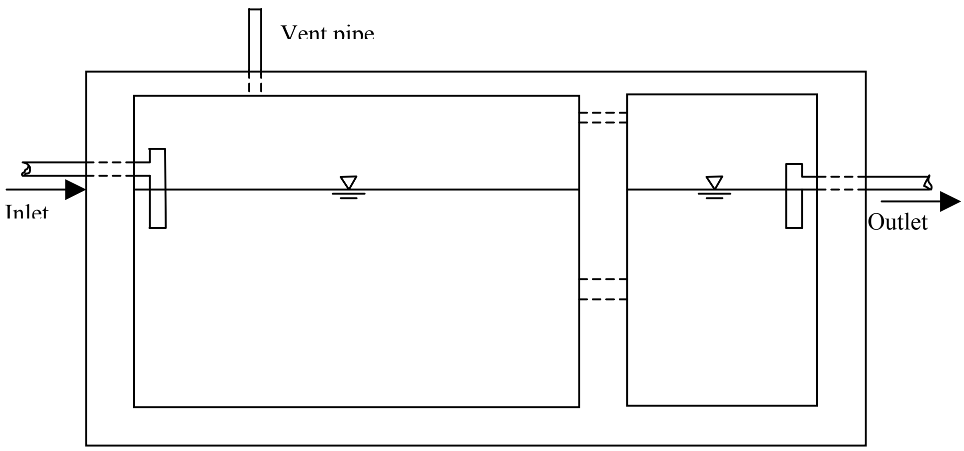

Figure 2-1 Schematic section of a septic tank.

[11] Although hydrogen sulphide is produced in septic tanks, odours are not usually a problem as the hydrogen sulphide combines with the metals in the accumulated solids to form insoluble metallic sulphides (Metcalf and Eddy, 1991)

2.2.3. Design

The key design considerations for septic tanks comprise determination of tank volume, tank geometry, inlet and outlet arrangements, number of compartments, and siting of the system.

Typically, tanks are designed for a 24-hour liquid retention time at maximum sludge depth and scum accumulation, although tanks with lower hydraulic retention times are used in some instances. It is commonly assumed that, at this stage, accumulated scum and sludge each occupy a third of the tank volume, leaving the balance third for liquid. Alternatively, the required scum and sludge storage volume is computed based on the anticipated frequency of desludging, and added on to the required liquid volume. In many countries, regulatory bodies recommend tank volumes for individual homes based on the number of bedrooms. The use of inlet tees reduces the turbulence in the tank and prevents break-up of the scum layer. Outlet tees, baffles and effluent filter vaults prevent the carry over of solids with the effluent. Multiple compartments of unequal size minimize inter-compartmental mixing.

Table 2-1 compares the different codes of practice and recommendations for the design of septic tanks in the US, UK, Ireland and Sri Lanka. The comparisons are made for septic tanks receiving black and grey water from single households, based on a desludging frequency of 1 year.

Table 2-1. Comparison of some Codes of Practice for septic tank design.

Parameter	US[12]	Ireland[13]	UK[14]	Sri Lanka[15]
Liquid volume, m^3	$3.0\ m^3$ up to 3 bedrooms $1.0\ m^3$ for each additional bedroom	$3.0\ m^3$ up to 3 bedrooms $0.5\ m^3$ for each additional bedroom	$2.7\ m^3$ up to 4 p.e. $0.18\ m^3$ for each additional p.e.	$0.11\ m^3$ per p.e. subject to a minimum of $1.0\ m^3$
No. of compartments	1 – 3	2	2	1 – 3
Volume ratio of compartments 1st : 2nd	2:1	2:1	2:1	2:1
Length: width	2:1 – 4:1	2:1 – 3:1	2:1	2:1 – 4:1
Minimum liquid depth,	0.3 m	1.0 m	1.5 m	1.0 m
Head space,	0.25 – 0.3m	0.3 m	0.3 m	-
Inlet – outlet level difference	25 – 100 mm	75 mm (minimum)	25 mm (minimum)	75 mm (minimum)

In cases other than single households, tank volumes need to be designed individually, based on flow rate and hydraulic retention time. In Sri Lanka, however, most septic tanks are not designed at all in an Engineering sense, and previous designs are duplicated blindly over and over again irrespective of the local site conditions. Although a Code of Practice for the design of septic tanks in Sri Lanka was introduced in 1986, few designs in the field are based on this and it remains largely ignored. Perhaps, the more rational approach to septic tank design for Sri Lanka would be an adaptation of the method described by Mara, which is based on the

[12] Adapted from Metcalf and Eddy, 1991.

[13] National Standards Authority of Ireland, 1991.

[14] British Code of Practice CP 302, 1972.

[15] Sri Lanka Standards Institution, 1986.

Brazilian septic tank code (Mara, 1996). This approach is discussed in more detail subsequently.

2.2.4. Setback distances

Since septic tanks are supposed to be water tight and buried, minimum setback distances are not particularly significant from either an aesthetic or environmental standpoint. All that is required is to exercise sound Engineering judgment in siting the tank properly so as not to affect the stability or structural integrity of the tank or any nearby feature. Setback distances, however, are important for disposal fields, soakage pits etc., which are commonly used for the disposal of septic tank effluents. Nevertheless, some local authorities stipulate minimum setback distances for the tanks themselves. For example, a minimum distance of 7 metres from the dwelling served and 20 metres from any other dwelling is required in Ireland (National Standards Authority of Ireland, 1991). Table 2-2 lists some examples of setback distances for septic tanks in the US.

Table 2-2. Some examples of setback requirements for septic tanks in the US.

Features	Setback distance (m)
Buildings	1.5
Property lines	3.0
Wells	15
Creeks or streams	15
Cuts or embankments	15
Pools	7.5
Water lines	3.0
Walks and drives	1.5
Large trees	3.0

(Adapted from Carter and Knox, 1985)

2.2.5. Materials and construction

Most septic tanks in developed countries are pre-cast or pre-fabricated in reinforced concrete, fiberglass or polyethylene. Concrete tanks are waterproofed internally and externally. In Sri Lanka, almost all septic tanks are built in-situ, usually of reinforced concrete or lined brick masonry. Regardless of the material and method of construction, a septic tank should be watertight and structurally sound. Some pre-cast concrete septic tanks are available for sale in Sri Lanka. However, their design and construction is not according to any particular standard, and their efficacy is uncertain. They are usually cylindrical, and are installed with their axes either vertical or horizontal.

2.2.6. Influent characteristics and loading

The influent characteristics of septic tanks depend on the type of application and differ considerably from that of municipal sewage. The influent characteristics of septic tanks serving individual households would vary depending on whether they are to handle black water, grey water or both. The lifestyle of the community served would also have an effect. The use of washing machines, dishwashers and kitchen waste grinders would have a significant effect on the influent quality as well as the flow rate. Even in the US there is a large disparity in the reported wastewater characteristics from individual households as is illustrated in Table 2-3.

Table 2-3. Comparison of 'typical' wastewater characteristics from individual residences in the US[16]

Constituent	Metcalf and Eddy, (1991)	Carter and Knox, (1985)
BOD_5 (mg/l)	392	300
SS (mg/l)	436	250
NH_3-N (mg/l)	14	12
Total P (mg/l)	19	25
Grease (Mg/l)	70	94
Total Coliform (cfu/100 ml)	10^8	2×10^6
Faecal coliform (cfu/100 ml)	-	3×10^4

Flow variation

One of the important considerations for small wastewater treatment systems is the large variation in flow rate. Flow rates from individual residences can vary from no flow, to as much as eight times average daily flow or more, depending on the time of day. This peak factor decreases gradually with increase in the number of users, and for systems serving 50 households or more, can be expected to decrease to around 1.6 (Metcalf and Eddy, 1991, Watanabe et al, 1993). The hourly peak factor needs to be taken into account, when designing septic tanks, in order to ensure proper settling of solids. Metcalf and Eddy recommend hourly peak factors ranging from 4 – 8 for individual residences, 6 – 10 for small commercial establishments, and 3 – 6 for small communities.

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[16] As reported by Metcalf and Eddy (1991) and Carter and Knox (1985). Both sets of data are for combined black and grey water from individual residences excluding contributions from ground kitchen wastes. Carter and Knox give weighted values of reported data from five different investigators.

The Indian practice is to base the flow on the number of ‘fixture units’. Each contributory ‘fixture’ in the building (i.e. urinal, washbasin, shower, sink, etc.) is assigned a weighted value, and the total flow is estimated based on the total number of weighted units. However, at present, important water-consuming appliances such as washing machines and dishwashers are not included in the weighting system (Indian Standards Institution, 1985). In the method described by Mara (Mara, 1996), the peak factor is taken into account by a simple empirical equation, which decreases the design effective hydraulic retention time, logarithmically, with increasing flow. This results in a decline of HRT from around 0.6 days at 5 p.e, down to a minimum floor value of 0.2 days at around 135 p.e. and beyond. This is a simple and effective way of accounting for the peak factor.

In addition to the peak hourly flow, the average daily flow is also dependent on the specific local conditions. This is more significant in developing countries where there is a much wider local variation of socio-economic conditions and lifestyles. In the US, wastewater flow rates from individual residences varies between 150 – 380 l/cap/d with a typical value of 210 l/cap/d used for design (Metcalf and Eddy, 1991, Carter and Knox, 1985). In Sri Lanka, the code of practice for septic tanks recommends a value of 120 l/cap/d for black and grey water combined and 40 l/cap/d for black water only. This is based on the national average water consumption figure, which is 160 l/cap/d. The combined black and grey water is assumed to be 75 percent of this figure, and black water only, is assumed to be 25 percent. The recommended values are probably rather low, for the urban and suburban context, where per capita water consumption is significantly higher than the national average and is around 200 – 250 l/cap/d depending on lifestyle. The corresponding wastewater discharge would be between 160 and 200 l/cap/d.

2.2.7. Effluent characteristics and treatment efficiency

The effluent characteristics and treatment efficiency of septic tanks alone have not been studied extensively, and published information is scarce. This is mainly due to the fact that septic tanks are commonly associated with leach fields and absorption trenches and they are usually studied together, as a single entity. Also, the effluent quality of septic tanks could be expected to be highly variable due to the diversity of influent conditions and design as mentioned earlier. Carter and Knox (1985) have attempted to present 'typical' physical and chemical characteristics of septic tank effluents based on information reported from a total of 41 tanks in the US. The values are shown in Table 2-4, and should be treated with some caution.

Table 2-4. 'Typical' physical and chemical parameters of septic tank effluents.

Constituent	Concentration (mg/l)
BOD_5	140
COD	300
Suspended Solids	75
Total Nitrogen	40
Total Phosphorus	15

(From Carter and Knox, 1985, based on composite information from 41 septic tanks in the US.)

Long-term random sampling of effluent from a septic tank in Sri Lanka, receiving black water from a population of 50, has revealed an effluent BOD_5 variation between 20 – 300 mg/l, with a mean of 90 mg/l. The 95 percent confidence range was 65 – 120 mg/l (Corea and Gamage, 1997). Field experience in Sri Lanka indicates that effluent BOD_5 values in the range of 50 – 150 mg/l could be considered typical for well-designed septic tanks in good condition. Suspended solids are typically less than 50 mg/l in the effluent from a good dual chamber tank. Indicator bacteria counts in the range of 10^7 cfu/100ml for total coliforms and 10^6 cfu/100 ml for faecal coliforms have been reported (Carter and Knox, 1985), and seem reasonable.

2.2.8. Operation and maintenance

The operation and maintenance of septic tanks is very simple and a periodic (annual) inspection for structural integrity, leaks and sludge depth is usually all that is required. When the sludge accumulates to the maximum depth, the tanks should be emptied by a vacuum tanker and the septage removed. The required frequency of removal of septage depends on the design of the tank and the loading. Tanks are designed for various emptying frequencies varying from 1 – 15 years. Table 2-5 shows typical emptying frequencies for some countries in Europe.

Table 2-5. Typical desludging frequencies for septic tanks in Europe.

Country	Desludging frequency
Italy	3 months – 1 year
Belgium	6 months – 1 year
UK	1 year
Germany	1 year
Switzerland	1 year
France	5 years max.

(From Philip et al, 1993)

Too frequent emptying of a septic tank inhibits the development of methanogenic bacteria and retards the digestion of sludge in the tank. Thus, tanks, which are designed for frequent emptying, would experience a higher sludge accumulation rate. Studies in France have revealed that methanogenesis is only completely effective after 2 years of operation and tanks should not be emptied before this period (Philip et al, 1993). The sludge accumulation rate drops rapidly after the first two years to around 0.22 l/cap/d from an initial 0.35 l/cap/d at start up. Philip et al recommend designing for a minimum 5-year emptying cycle based on a sludge accumulation rate of 0.2 l/cap/d. This is in general agreement with Mara (1996), except for the fact that he recommends an accumulation rate of 0.1 l/cap/d for emptying cycles greater than five years. This latter value is probably better suited to Sri Lanka, which has a climate closer to Brazil from where the figure originates. The efficiency of anaerobic digestion is known to increase sharply above 15^0C (Mara, 1996).

Considering the fact that desludging is the single major operational cost of a septic tank and that handling and disposal of septage, which is highly polluting in comparison to sewage, is a potentially expensive problem for local authorities, it is desirable to maximize sludge digestion within the tank and reduce emptying frequency. In Sri Lanka, where the cost of emptying a septic tank by vacuum tanker is relatively high, the average householder would probably consider an emptying frequency of less than five years to be unacceptable.

Septic tanks should not be emptied completely during desludging as is the common practice, as the hydrolytic enzymes associated with biological activity in septic tanks have been found mainly bound to the insoluble sludge particles in the tank (Philip et al, 1993). Some amount of sludge should be left in the tank as seed for the next cycle of operation.

2.2.9. Advantages and disadvantages

The main advantages of septic tanks as primary treatment units for on-site applications could be summarized as follows.

- Low-cost, low maintenance, nuisance free operation
- Long term reliability if designed and installed appropriately
- Non land-intensive, minimal setback requirements
- Simple construction with locally available materials and skills
- Good primary treatment capable of settling and digestion of sludge, with very low sludge production.

In addition to the above advantages, septic tanks could also be used in conjunction with settled sewer schemes in the development of urban sanitary infrastructure.

The main disadvantages of septic tanks are the problems associated with inappropriate design and application. These include poor sizing, dimensioning and location of the tanks, poorly designed and placed inlets and outlets, poor construction etc. Users also tend to neglect desludging the tanks until they fill up completely with sludge, leading to failure of the system. It should be noted, however, that most of the traditionally perceived drawbacks of the septic tank systems are actually associated with the effluent disposal fields and soakage pits and not with the tanks themselves.

2.3. Anaerobic filters

2.3.1. Background

Although most modern literature on the subject attributes the development of the Anaerobic filter to either Coulter et al in 1957 or Young in 1968 (Kobayashi et al, 1983, Young, 1991), it is evident that anaerobic filters have been used for sewage treatment in the UK well before the turn of the century. The first field-scale anaerobic filter in the UK is credited to Scott Moncrieff, who in 1891, used a "continuous upward flow tank filled with coarse flints to treat sewage from a household of ten persons"[17] (Barwise, 1901). Subsequently, septic tanks were used as a primary treatment step, prior to the anaerobic filter, to avoid problems of filter clogging. By the turn of the century, consequent to an extensive review of the sewage treatment works in Manchester, Chesterfield and Leeds, as well as other places in the UK, it was recommended that sewage should be treated by septic tanks followed by anaerobic filters prior to tertiary treatment by infiltration-percolation beds or trickling filters for nitrification (Barwise, 1901)[18].

In modern times, anaerobic filters have mainly been used to treat medium to high strength wastes, typically with BOD values in excess of 1000 mg/l. Modern applications of the anaerobic filter for low-strength domestic sewage treatment have been limited. However, some work has been done in this area in the recent past, and interest is gradually developing in its potential for low-cost sewage treatment (Kobayashi et al, 1983, Watanabe et al, 1993, Panswad and Komolmothee, 1997, etc.). In this study, the main interest in the anaerobic filter is in its potential as a low-cost, low-maintenance, secondary treatment unit process to treat septic tank effluents in Sri Lanka.

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[17] It is interesting to note that this occurred five years before the septic tank was first introduced to Britain.

[18] Anaerobic filters were referred to as anaerobic 'bacteria beds' at the time.

2.3.2. Physical description

The anaerobic filter basically comprises a watertight tank containing a bed of submerged media. The filter media retains the biological solids within the reactor as a fixed film attached to the media, as solids trapped within the interstices of the media, or as solids suspended within or beneath the media in the form of a granulated or flocculated sludge mass (Young, 1991). The wastewater flow through the media is continuous, and usually, upflow. As the wastewater flows through the filter, it comes into contact with the biomass in the filter and is subject to anaerobic decomposition. The particular advantage of preceding an anaerobic filter with a septic tank is that the first stages of anaerobic decomposition would take place in the septic tank and the influent to the filter would consist mainly of volatile fatty acids. Therefore the predominant treatment step in the filter would be the final methanogenesis. This would allocate the long solids retention time provided by the filter specifically for the slow-growing methanogens, which need it most, with minimal competition from the fast growing acidogens and acetogens which would predominate in the septic tank. Figure 2-2 shows a schematic section through a typical anaerobic filter with crushed rock as filter media.

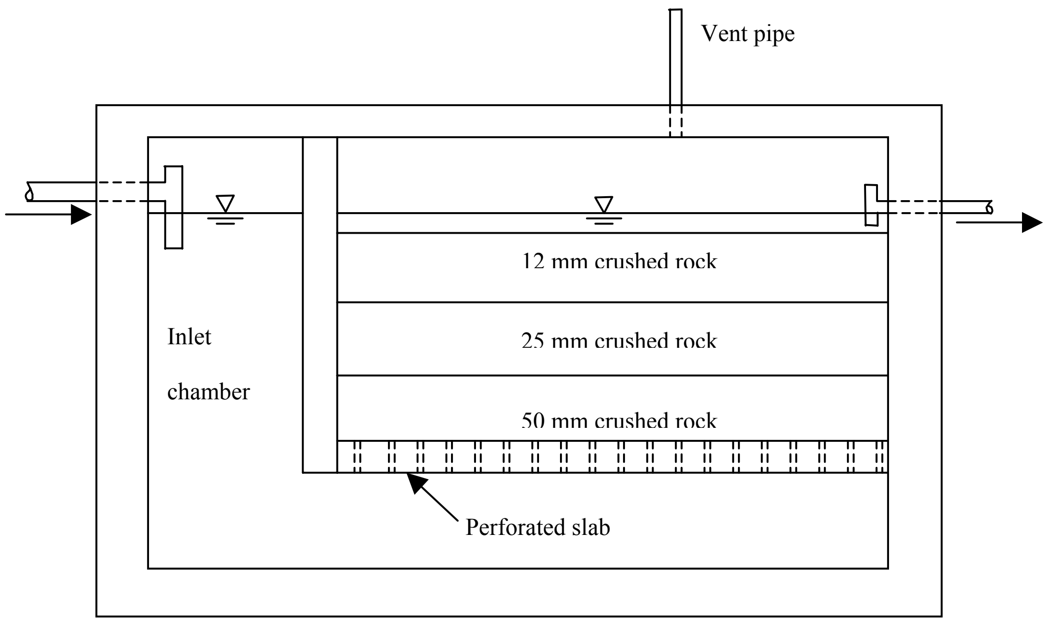

Figure 2-2 Schematic section of an anaerobic filter.

2.3.3. Design

No clear guidelines exist for the design of anaerobic filters in recent literature, particularly for domestic sewage treatment. In 1901, Barwise recommended the use of septic tanks with 12 hour hydraulic retention times followed by anaerobic filters of 12 hour HRT, for the treatment of domestic sewage. He further, specified filter media of ‘coke breeze, clinker, or other hard material’ of size between 1 –3 inches (Barwise, 1901). Based on the results of full-scale trials conducted at Chesterfield, he projected fifty percent efficiency in treating organic matter by the filter unit. Young (1991) reviewed the performance of 30 full scale and large pilot scale anaerobic filters in the USA, Canada and Europe, and concluded that hydraulic retention time is the single most important design and performance parameter. He found that influent waste strength and reactor height have no effect on treatment efficiency, and that media specific surface and orientation have only a minor effect. However, most of the systems he studied were treating high strength wastes with COD above 1500 mg/l. The same need not necessarily apply for low strength domestic sewage. In Thailand, Panswad and Komolmothee (1997) reported a linear relationship between BOD removal and influent BOD concentration for a combined septic tank-anaerobic filter unit treating municipal sewage with BOD_5 around 150 mg/l.

Based on pilot scale studies on septic tank effluents in Sri Lanka (Corea and Parameshwaran, 1994), several anaerobic filters using rock media have been designed and installed to treat septic tank effluents in Sri Lanka. A hydraulic retention time of approximately 18 hours was typical, and long term monitoring indicated that the filters were performing satisfactorily (Corea et al, 1998).

2.3.4. Setback distances

Local experience in Sri Lanka has shown that no setback distance is required for anaerobic filter units other than that which may be necessary for the structural stability and integrity of the units themselves and any adjacent features. No detectable odours have been reported even within one metre distance of either the unit, or the discharge point, and since the units are buried, visual impact is minimal.

2.3.5. Materials and construction

Anaerobic filters can be constructed in much the same way as septic tanks. Some pre-fabricated units integrated with a septic tank are now available in Thailand. However, in Sri Lanka, they are constructed in-situ, out of reinforced concrete or brick masonry. The filter media is crushed rock, which is washed free of sand and grit before placing. The filter floor is made up of perforated reinforced concrete slabs to evenly distribute the flow across the filter. This perforated floor slab is a significant component of the cost of the unit, due to the fact that it needs to be sufficiently strong to support the rock media.

2.3.6. Influent characteristics and loading

Since the anaerobic filters would receive septic tank effluent, the influent characteristics would be similar to that of septic tank effluents, the characteristics of which have been discussed previously in section 2.2.7. For Sri Lanka, typically, 50 – 150 mg/l BOD_5 and less than 50 mg/l SS could be assumed, and in the absence of better information, total nitrogen, total phosphorus and indicator bacteria levels could be assumed to be as given in Table 2-4 in section 2.2.7.

Influent hydraulic loading to the anaerobic filter could be expected to be highly variable. The high instantaneous peak flow experienced by the septic tank could be assumed to pass on to the filter with minimal hydraulic buffering. Nevertheless, anaerobic filters have been found to be highly resistant to hydraulic shock loads and variations in influent quality (Panswad and Komolmothee, 1997, Kobayashi et al, 1983, Watanabe et al, 1993, Young, 1991).

2.3.7. Effluent characteristics and treatment efficiency

In previous literature, many authors have discounted the applicability of anaerobic filters for low-strength domestic sewage treatment due to the low treatment efficiencies reported: Typically in the order of 50 – 60 percent (Young, 1991). However, it should be noted that in the context of low-strength domestic sewage, particularly septic tank effluent, a treatment efficiency of 50 percent would often be sufficient to produce effluents with less than 30 mg/l BOD_5. Furthermore, in the range of operations under consideration, the treatment efficiency increases with increasing organic loading, and thereby tends to produce a consistent effluent quality under varying organic load (Panswad and Komolmothee, 1997, Corea et al 1998). In Brazil, Chernicharo and Machado (1998) reported a similar consistency of effluent quality in pilot scale anaerobic filters used to treat low strength domestic sewage effluent from an UASB reactor. Table 2-6 shows a comparison of published data on effluent quality and treatment.

Table 2-6. Comparison of anaerobic filter performance for domestic sewage treatment.

Investigator	**Temp, °C**	**HRT, hrs.**	**Influent BOD, mg/l**		**Effluent BOD, mg/l**		**Effluent SS, mg/l**
			range	mean	range	mean	mean
Corea et al (1998)	27	18	20-297	90	5-44	24	<25
Chernicharo and Machado (1998)	-	12 6 5 3 2 1.5	-	55 30 21 24 45 48	-	31 27 18 21 22 27	19 10 9 9 18 21
Kobayashi et al (1983)	20-35	24	44-573	163	13-97	40	32
Panswad and Komolmothee (1997)	25	30 15 10	- - -	- - -	- - -	27 46 56	17 36 44
Harris[19]	15-25	12-18	100-150	-	60-70	-	-

[19] As quoted by Young, 1991

No significant removal of nitrogen or phosphorus should be expected in anaerobic filters using crushed rock (gneiss) or plastic as a filter medium. Panswad and Komolmothee (1997) reported removal efficiencies of less than 10 percent for total nitrogen, ammonia nitrogen and total phosphorus in a combined septic tank-anaerobic filter unit treating domestic sewage. Maximum removal of organic nitrogen was 35 percent. Specific removal of phosphorus has been reported in anaerobic filters using iron contactors as packing medium (Choung and Jeong, 2000). A single log unit removal of faecal coliforms has been reported (Kobayashi et al, 1983), which is not particularly significant as far as pathogen removal is concerned. Some authors report a significant drop in performance at temperatures below 20°C.

2.3.8. Operation and maintenance

Anaerobic filters used in the current context are essentially self-operating, with no significant maintenance requirements beyond an annual inspection. Mara (1996) mentions the likely requirement of draining the filter and flushing it with water once every two years or so. However, field experience suggests that filters designed for 18-hour retention times would probably run for about 5 years or more before this becomes necessary.

2.3.9. Advantages and disadvantages

Anaerobic filters have a good potential for secondary treatment of septic tank effluents, either on-site, or semi-collectively for population equivalents up to, perhaps, 300. They can also be used in conjunction with other unit processes in order to treat wastewater for on-site re-use. In Sri Lanka, where ambient temperatures in most areas are constantly close to 30°C they have a high applicability, though some precaution should be exercised when they are located in the high mountain areas where temperatures could drop below 20°C. The main advantages of the filters are their low-cost, which is typically in the same order as that of a septic tank, with low-maintenance, nuisance-free operation producing consistent good quality secondary effluent. They are resilient to the sudden variations in flow and hydraulic loading typical of small systems. Their construction is simple, with locally available materials and skills, and they have negligible production of secondary sludge.

The main disadvantage of anaerobic filters is their inability to remove nutrients and pathogens and the lack of proper design guidelines.

2.4. Horizontal flow reed beds

2.4.1. Background

Constructed wetlands systems have been widely used in the US, Europe and Australia for a wide range of applications, in many forms. These have been mainly large land-intensive schemes often covering many hectares in extent (Kadlec and Knight, 1996) and would not normally find a place in urban sewage treatment applications. The main applications in the UK have been limited to tertiary treatment and polishing of secondary effluents to meet increasingly stringent discharge standards. However, in the US, wetlands have also been widely used for secondary treatment of sewage. More recently, a limited amount of literature has emerged with regard to subsurface flow constructed wetlands, or reed beds, in small on-site sewage treatment applications, and in some cases, specifically for the treatment of septic tank effluents (Perfler and Haber, 1993, Netter, 1993, Stott et al, 1997, Philippi et al, 1999, Mander et al, 2000, Srinivasan, et al, 2000). In particular, some evidence suggests that for warm climates, reed bed systems may not necessarily be as land intensive as previously supposed. This together with the fact that they have a potential for effluent treatment for reuse (House et al, 1999), make them worthy of consideration. Reed bed systems are also referred to as plant rock filters, root zone method, gravel bed hydroponic filters, vegetated submerged beds etc. by various authors.

2.4.2. Physical description

Typically, reed beds comprise a long channel lined with impermeable material and filled with sand, gravel, stone, or soil media which supports a stand of emergent vegetation - commonly reeds. The wastewater enters the bed through an inlet zone devoid of vegetation and flows horizontally through the bed with the water surface wholly contained below the media surface. The effluent is collected via an outlet zone and then discharged. The water level in the bed is controlled by a simple swivelling - elbow device at the outlet.

Microbial growth occurs attached to the media surface, the plant roots and stems and suspended within the interstices of the bed. Oxygen is transferred to the root zone of the bed via the plant rhizomes. This creates alternating aerobic and anaerobic sites within the bed, which is a unique feature of these systems. A complex combination of sedimentation, adsorption, precipitation, filtration and biodegradation are thought to occur within the bed. Reed bed systems have been credited with removal of organic matter, suspended solids, nutrients, heavy metals, trace elements, refractory organics and pathogens at varying levels by various investigators (Kadlec and Knight, 1996). Figure 2-3 shows a schematic section through a typical reed bed.

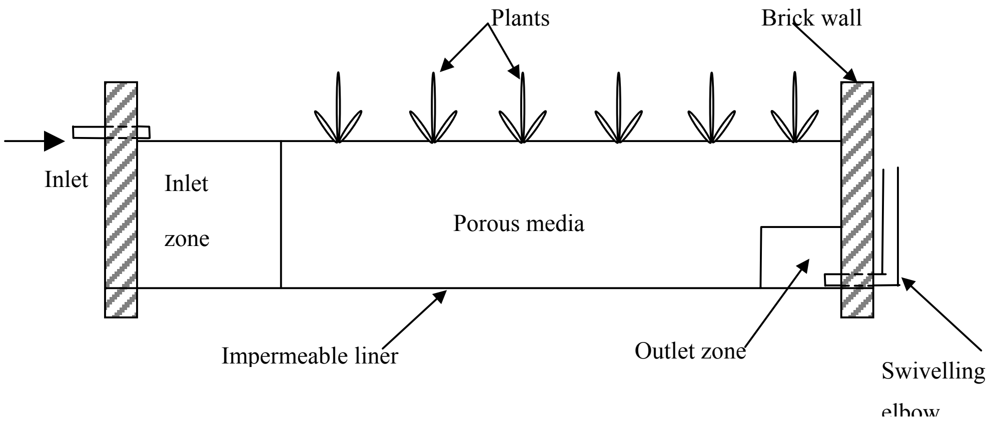

Figure 2-3 Schematic section through a reed bed.

2.4.3. Design

The design of reed bed systems involves the selection of media and vegetation and determination of the size and geometry of the bed. The bed size would be dependent on the specific treatment objectives. Though many design approaches and models have been proposed, they are of limited practical value due to the highly variable nature of these systems.

For practical applications in Sri Lanka, bed size could be determined based on population equivalent (or flow). Most of the other details could be standardized. Water depth could be standardized at 0.6m, based on plant root penetration of common reeds (*Phragmites spp.*) as proposed by Reed et al, (1988). Although specific area values of 5 m^2/p.e. (Perfler and Haber, 1993) and 8 – 10 m^2/p.e. (Netter, 1993) have been proposed for the treatment of septic tank effluents in Austria and Germany respectively, much lower values could be used in Sri Lanka. This is confirmed by detailed studies done in Egypt, where good results have been obtained at field scale, with values as low as 0.66 m^2/p.e. (Williams et al, 1995).

Kadlec and Knight (1996) provide an exhaustively comprehensive review of current information on wetland systems, based on over 200 wetland systems in North America and 500 subsurface flow systems in Europe and the UK. A critical evaluation of the information presented in this work reveals that a first order plug flow model is the best practical design approach for reed beds, and that the kinetic rate constant is dependent mainly on temperature and bed porosity. Subsequent published literature on the subject does little to change the validity of these conclusions (Vymazal et al, 1998, Laber et al, 1999).

Consider a simple design calculation based on the method advocated by Reed et al (1988) and Metcalf and Eddy (1991). Reed advocates a first order plug flow model given by

$$C_e/C_o = \exp(-K_T.\ A_s.d.n)/Q \quad -(2.1)$$

Where,
C_e = effluent BOD, mg/l
C_o = influent BOD, mg/l
K_T = first order temperature dependent rate constant, $days^{-1}$
A_s = surface area of the bed, m^2
n = bed porosity as a decimal fraction
d = water depth, m
Q = average flow, m^3/d

and

$$K_T = K_{20}.(1.06)^{(T-20)} \quad -(2.2)$$

Where K_{20} varies with bed porosity in the manner

$$K_{20} = K_0.(37.31\ n^{4.172}) \qquad \text{-(2.3)}$$

K_0 is the "optimum" rate constant for a fully developed root zone, and for typical domestic sewage, has a value of 1.839 d^{-1} (Reed et al, 1988).

For a hypothetical case in Sri Lanka, let

C_e = 10 mg/l BOD_5 (desired effluent quality, say)
C_o = 100 mg/ BOD_5
n = 0.4 for coarse sand beds and gravel beds planted with *Phragmites* (Metcalf and Eddy, 1991, Kadlec and Knight, 1996)
T = 27°C
d = 0.6 m
Q = 7.2 m^3/d (for a p.e. of 50, say)

Substituting these values in the model (Eqn. 2.1) gives the required surface area,
A_s = 31 m^2.

This corresponds to a specific area of 0.6 m^2/p.e. and a hydraulic retention time of 1 day, where HRT is given by

$$HRT = nA_sd/Q \qquad \text{-(2.4)}$$

These values are in very close agreement with the experimental results obtained in Egypt by Williams et al (1995), who obtained comparable removals at a hydraulic loading of 0.66 m^2/p.e. under similar conditions. Therefore, specific area requirements for reed beds in Sri Lanka could reasonably be estimated by this method.

The relative importance of reed beds in the present context is not so much for BOD and suspended solids removal, as for nutrient and pathogen removal. However, the experience from Egypt indicates that both ammonia nitrogen and total nitrogen removal is associated with BOD and suspended solids removal[20]. Faecal coliform counts, too, exhibit a correlation with BOD. This reinforces the views of Gray (1989) and Gersberg et al (1989) that the predominant coliform removal mechanisms in reed beds are similar to that of BOD removal, i.e., by biofilm adsorption. It is significant, however, that similar reed bed systems in the UK investigated by Williams et al showed much poorer pathogen removal at even less than half the hydraulic loading rate, implying that pathogen removal is highly temperature sensitive. Williams et al (1995) also found that log unit reduction of coliforms progressed in a near linear manner along the length of the bed, prompting them to propose a 'decimal reduction distance' (DRD) for the beds. The DRD being the length of bed required per log unit removal of coliforms.

In addition to bed area, a maximum bed length is applicable in order to prevent flooding of the downstream parts of the bed due to a rising phreatic surface. A maximum bottom slope is also applicable to prevent bed dry-out due to a declining phreatic surface. Reed et al (1988) also suggest a maximum horizontal flow velocity for reed beds of 8.6 m/d. This would imply a minimum width restriction for the bed.

2.4.4. Setback distances

It is not possible, given the present state of literature, to stipulate guidelines for setback distances for reed bed systems, and they should be determined according to local site conditions. However, the aesthetic nuisance levels of reed beds could be expected to be much less than those of free water surface and floating aquatic plant wetlands (Kadlec and Knight, 1996). Therefore, setback distances could be minimal.

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[20] However, a 'threshold flow rate' exists (13.5 mm/d in this case), beyond which there is a sharp increase in effluent ammonia nitrogen.

2.4.5. Materials and construction

The materials required for reed beds are relatively simple. The beds need to be made impermeable and an approach similar to that of waste stabilization ponds, outlined by Mara (1997) could be adopted. If synthetic membrane liners are to be used, 2mm smooth plastic or butyl rubber is preferred (Reed et al, 1988). All materials are either locally available, or could be fabricated locally. Plant stock could be obtained from natural reed stands, and rhizomes should initially be established approximately one metre apart (Kadlec and Knight, 1996).

2.4.6. Influent characteristics and loading

Influent characteristics and loading would depend on the particular application and the pre-treatment process. In general, pre-treatment by at least a septic tank would be desirable in Sri Lanka.

2.4.7. Effluent characteristics and treatment efficiency

A large body of literature is available on effluent quality and treatment of reed bed systems (Kadlec and Knight, 1996, Geller, 1997, Batchelor and Loots, 1997, Srinivasan et al, 2000). However, for Sri Lanka, the field scale results obtained in Egypt by Williams et al (1995) would probably be the most representative.

Table 2-7 summarizes the effluent quality and treatment that could be reasonably expected for secondary treatment of domestic sewage (septic tank effluent) for a case similar to that of the design example in section 2.4.3.

Table 2-7. Typical treatment that could be expected by reed beds for secondary treatment of septic tank effluents in Sri Lanka.

Parameter	Influent	Effluent
BOD_5, mg/l	90	<15
SS, mg/l	70	<15
NH_3-N, mg/l	20	3
Total N, mg/l	0.3	3
Faecal coliforms, cfu/100ml	10^5	10^2

(Based on the findings of Williams et al, 1995)

Phosphorus removal is a complex and variable process, and is dependent on the type of media used. Media type could be selected to specifically target phosphorus removal. However, in general, for gravel and rock bed systems, an effluent concentration of 4 – 6 mg/l Phosphorus (corresponding to approximately 10 –20 percent removal) could be considered typical (Reed et al, 1988, Kadlec and Knight 1996).

Further studies of the full-scale systems in Egypt (Stott et al, 1997) have demonstrated excellent removal of all types of helminth eggs including *Ascaris, Trichuris, Hymenolepsis and Toxocara spp.* Protozoal cysts, including *Giardia* cysts were also removed by the reed bed systems. Virus removals of, typically, 2 log units have been reported for reed bed systems (Reed et al, 1988, Williams et al 1995).

2.4.8. Operation and maintenance

The operation and maintenance aspects of reed bed systems are slightly more frequent than septic tanks and anaerobic filters. However, they remain simple and inexpensive. The main features are the maintenance of the vegetation and monitoring of the water level. This includes thinning of vegetation, filling of vacancies etc., which could easily be handled locally.

2.4.9. Advantages and disadvantages

Reed beds provide enhanced treatment with potential for effluent reuse. They are slightly more land-intensive than anaerobic filters and need more regular attention and maintenance. The land requirement is approximately 30 percent more than an anaerobic filter for the same population equivalent. However, unlike anaerobic filters which can be buried and the land above put to limited use such as parking etc., reed beds are essentially surface features, which adds to the overall opportunity cost of the land. Reed beds also have a greater visual impact than anaerobic filters.

2.5. Infiltration-percolation beds

2.5.1. Background

Infiltration-percolation beds have been widely used for on-site treatment of septic tank effluent in Europe and the US for quite some time. They have been the traditionally favoured option in situations where local site conditions preclude the use of seepage fields and trenches. The ones in use today are essentially the same as those used as far back as 1868 (Metcalf and Eddy, 1991, Barwise, 1901). They are capable of efficient removal of organic matter and suspended solids as well as nutrients and pathogens (Brissaud and Lesavre, 1993, Boller et al, 1993, Salgot et al, 1996). They are also referred to as subsurface bio-filters, biological sand filters, intermittent sand filters, gravel percolation beds etc. by different authors.

2.5.2. Physical description

Typically, infiltration percolation beds are shallow beds of sand, 600-700 mm deep, provided with a surface distribution system and an underdrain system. Septic tank effluent is periodically applied to the surface of the sand bed and the wastewater percolates down through the bed. Treated effluent is collected by the underdrain system. Most infiltration-percolation beds are buried, although, sometimes, open filters are also used.

The treatment mechanisms in infiltration-percolation beds are, to an extent, similar to that of reed beds. BOD removal and nitirification is achieved mainly by aerobic biological growth within the filter bed. Denitrification occurs through anaerobic bacteria, which exist in anaerobic micro-environments within the bed. Other constituents are removed by chemical and physical sorption (Metcalf and Eddy, 1991). Aerobic conditions are maintained either by the intermittent application of wastewater, or by venting the underdrain system, or both. Sometimes for larger flows from commercial establishments and small communities, the effluent is recycled through the filter for improved treatment. These are sometimes called recirculating flow filters, and the media size is often larger than the flow-through systems. Figure 2-4 shows a schematic section through a typical percolation bed.

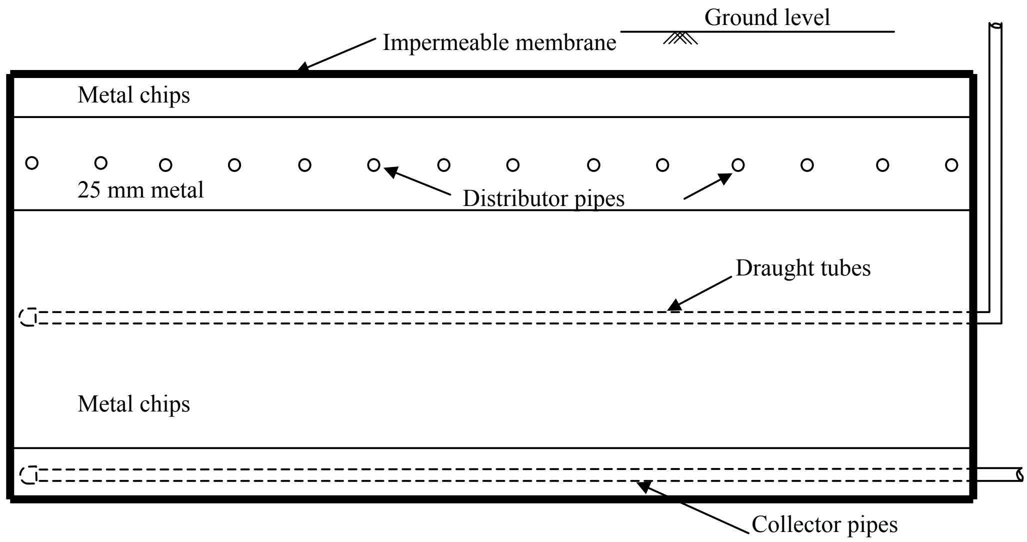

Figure 2-4 Schematic section through a buried percolation bed.

2.5.3. Design

The design of infiltration-percolation beds involves determining the size of the filter and designing the influent distribution system and the underdrains. The design process is relatively simple and standard (Metcalf and Eddy, 1991). However, there is a fairly large variation in the specific sizing of the beds and values ranging from 1 – 10 m^2/p.e. have been used in a rather ad-hoc manner (Guilloteau et al, 1993, Boller et al, 1993). Although a fairly large body of literature exists on the subject, most of the publications on field scale units are mainly reviews of performance and operation, and very little attention has been paid towards rationalizing the design for future systems. However, some systematic work done in France by applying pilot scale results to full-scale systems (Fazio et al, 1993), has revealed that good removal of organic matter, suspended solids and pathogens, together with good nitrification can be achieved for septic tank effluents, using a specific area of 1 m^2/p.e. The systems have performed well in the medium term (two years). If pathogen removal is not a treatment objective, the specific area could be further reduced to 0.5m^2/p.e. (Guilloteau et al, 1993, Boller et al, 1993).

For passive aeration systems, alternate periods of operation and resting are sometimes required to renew aerobic conditions. Equal periods of resting and operation are commonly used which effectively doubles the land requirement. Guilloteau et al (1993) reported that the rest periods should ideally be double the operational periods, which would increase the land requirement even more. However, by careful design of the influent distribution system, continuous operation without resting is possible (Fazio et al, 1993). Netter et al (1993) have developed a full-scale horizontal flow variant with low-intensity aeration (1 m^3/hr). This reduces the specific volumes to 0.6m^3/p.e. for septic tank effluent. Latvala (1994) proposed the use of multiple layered influent-feeding at intervals of 30 cm down the bed – a sort of 'multi-storeyed' arrangement. This was based on the fact that the major portion of the treatment occurs in the first 15 cm of the bed (Guilloteau et al, 1993). Pilot plant studies indicate the possibility of reducing specific area requirements down to 0.3 m^2/p.e. However, pathogen removal is unlikely in this case and no field scale data is available as yet. In Sri Lanka, values as low as 0.11 m^2/p.e. have been used for tertiary treatment with, apparently, satisfactory performance (Corea et al, 1998).

For Sri Lanka, a balance would have to be made between active aeration with lower land intensity and passive aeration with higher land intensity, depending on land availability and maintenance capability of the specific cases. However, since passive aeration systems can be buried, the land above could be put to limited use, reducing the opportunity cost of the land.

2.5.4. Setback distance

For buried systems, no significant setback distances apply, and units can be located even directly in the front yards of homes (Metcalf and Eddy, 1991). However, if open systems are used, odours could occur if the beds become flooded and some safe setback distance should be provided. No specific guidelines are available in literature.

2.5.5. Materials and construction

Construction and materials are similar to reed bed systems, except for the pipe work involved in the influent distribution system and the underdrains. For Sri Lanka, the preferred choice would be uPVC pipes, which are produced locally. Boller et al (1993) recommend the use of 50 percent $CaCO_3$ (limestone) in the media to improve performance by pH buffering and this should be considered, particularly in the case of systems receiving kitchen and restaurant wastewater. If active aeration systems were to be used, either small centrifugal pumps or blowers would be required with the attendant switchgear, wiring etc. For beds with intermittent loading, some form of dosing siphons would be required. All these are freely available in Sri Lanka, either manufactured locally, or imported. Construction and installation requires a slightly higher level of skills than for the previous systems.

2.5.6. Influent characteristics and loading

In the case of secondary treatment applications, the influent to infiltration-percolation beds would typically be septic tank effluent, the characteristics and loadings of which have been described previously in section 2.3.6. In the event of tertiary treatment applications, Infiltration-percolation beds would most likely follow either anaerobic treatment units, or reed bed units. Consequently, the influents to the beds would either be anaerobic filter effluent or constructed wetland effluent. The characteristics of both have been discussed in sections 2.3.7 and 2.4.7 respectively.

2.5.7. Effluent characteristics and treatment efficiency

The effluent characteristics and treatment efficiency of infiltration-percolation beds will depend on the type of application and the specific treatment objectives for which the beds are designed. A fully nitrified effluent with BOD_5: SS less than 10:15 mg/l, and faecal coliforms less than 100 cfu/100ml can be practically achieved under field conditions (Fazio et al, 1993, Boller et al, 1993, Nielsen et al, 1993, Guilloteau et al, 1993). Although denitrification is cited with an overall total nitrogen removal of around 40 percent by some authors, it cannot be predicted with confidence, and according to the results obtained by Guilloteau et al (1993), is unlikely to occur in shallow beds at depths less than 50 cm. As in the case of reed beds, the overall treatment efficiencies of infiltration-percolation beds could reasonably be expected to increase with temperature in tropical climates. However no temperature related data is currently available.

2.5.8. Operation and maintenance

The operation and maintenance requirements of infiltration percolation beds are greater than any of the unit processes discussed previously. For beds designed for alternating operations and resting cycles, a reliable operator is required to open and shut the flow to the beds in the proper sequence. For pressurised systems, pumps or blowers need to be maintained, and would have an associated energy consumption. Dosing siphons, valves and plumbing fixtures would require basic maintenance. However, for systems designed to be fully passive and buried, i.e. passive aeration and continuous flow, the operation and maintenance requirements would be minimal.

2.5.9. Advantages and disadvantages

Infiltration-percolation beds have the potential to treat wastewater up to reuse standards, similar to that of reed beds, with a similar land requirement. Unlike reed beds however, they could be buried if necessary. Active systems would be slightly more expensive to maintain and operate than reed beds, and even though replacement parts and service for the mechanical equipment such as pumps, blowers and dosing siphons are available nationwide, the availability could be intermittent at times.

2.6. Vertical flow planted gravel filters (VGPGF's)

2.6.1. Background

Vertical flow planted gravel filters evolved during the field component of this study, and were initially developed in order to combine the advantages of horizontal flow reed beds with those of infiltration-percolation beds to achieve good treatment with a smaller overall footprint. Consequently, they are in essence, a hybrid of the two - a sort of vegetated percolation bed or vertical flow reed bed. They could be either secondary or tertiary treatment unit processes. Elsewhere, particularly in Europe, various vertical flow variants of reed beds have been receiving increasing attention and interest over the past decade or so. Cooper (1999) reviewed some of the vertical flow reed beds, which have been implemented in Europe over the past ten years and found that they can achieve secondary treatment with a specific area as little as 1 – 2 m^2/p.e. as opposed to 5 –10 m^2/p.e. required for horizontal flow reed beds under similar conditions. He attributes this to their greater oxygen transfer capacity. Unlike horizontal flow systems, where oxygen transfer to the bed is mainly by the plant roots, in vertical flow systems, oxygen is transferred mainly though mass transfer and diffusion through intermittent loading of the unsaturated bed (Platzer and Mauch, 1997).

2.6.2. Physical description

VFPGF's are, essentially, open infiltration-percolation beds, with the bed vegetated. The same influent distributor and underdrains are maintained, and plants are planted on the surface of the bed in between the distributor pipes. In pressurised systems where the influent is pumped through the distributor, it is sometimes sprayed upwards in the air a few feet, in order to aerate the influent before landing on the bed. This is usually done in the case of tertiary treatment VFPGF's. In secondary VFPGF's or in non-pressurised gravity feed systems, the distributor pipes are usually buried a few centimetres below the surface of the bed. The influent percolates through the bed and is collected via the under drains. Usually, no water level is maintained in the bed as in the case of horizontal flow reed beds. Figure 2-5 shows a schematic section through a typical VFPGF.

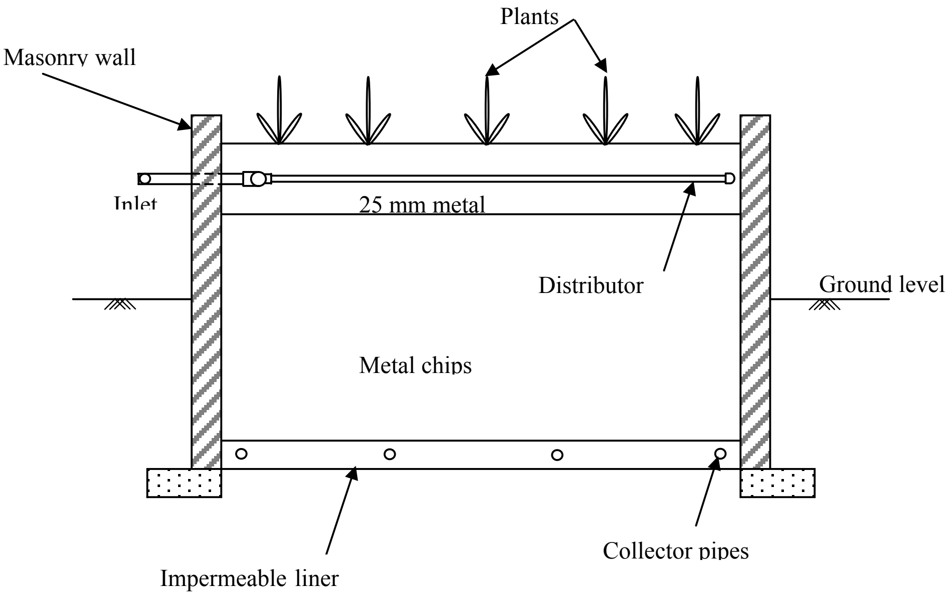

Figure 2-5 Schematic section through a VFPGF.

2.6.3. Design

The design of VFPGF's is similar to that of infiltration-percolation beds and is based on specific area. The specific area recommended for Europe is 1 – 2 m^2/p.e. for secondary treatment and 1 m^2/p.e. for tertiary treatment of Municipal wastewater (Cooper, 1999, Schonerklee et al, 1997). However, no consensus has been reached regarding the specific area criteria as yet. For small systems of less than 100 p.e. receiving septic tank effluent, Cooper (1999), recommends the use of two vertical flow beds in series sized as follows.

$$A_1 = 3.6P^{0.35} + 0.6P \qquad -(2.5)$$

and $A_2 = 0.5A_1$

Where, A_1 = Area of first vertical flow bed, m^2

A_2 = area of second bed, m^2

P = population equivalent

This results in a total specific area requirement decreasing from 3 m^2/p.e. for 4 p.e. down to 1.17 m^2/p.e.for 100 p.e.

2.6.4. Setback distances

Required setback distances for VFPGF's could be assumed to be similar to those of horizontal flow reed beds as discussed in section 2.5.5 above.

2.6.5. Materials and construction

Materials and construction of VFPGF's would be similar to that of infiltration-percolation beds, as discussed in section 2.5.5. The selection and placing of the upper layers of the bed, so that it doesn't clog, is considered essential to good operation of these systems (Cooper, 1999).

2.6.6. Influent characteristics and loading

Influent characteristics and loading would be similar to that of infiltration-percolation beds as described in section 2.5.6. Many authors recommend the use of multiple beds in parallel with intermittent loading resting cycles, typically in the order of weeks, in order to avoid solid clogging at the bed surface (Cooper et al, 1999 etc.) However, some authors have reported an increase in clogging with intermittent operation (Platzer and Mauch, 1997). Also, this has to be considered against the consequent doubling of the total system footprint, which in the urban and suburban context in Sri Lanka, may not be viable, and other options such as careful media selection and limiting the organic loading rate (Platzer and Mauch, 1997), may be required in order to avoid clogging.

2.6.7. Effluent characteristics and treatment efficiency

The published literature on vertical flow reed beds reports varying degrees of treatment efficiency. However, in general, the BOD and COD removal efficiencies of vertical flow beds are generally higher than equivalent horizontal flow reed beds, as is the nitrification efficiency and removal of pathogens. However, the removal of suspended solids in vertical flow beds is somewhat poorer than in horizontal flow beds (Cooper et al 1999). Typically BOD and COD removal efficiencies of between

85 – 98 percent have been reported (Schonerklee et al, 1997, Laber et al, 1999, Vymazal et al, 1998) for beds of around 1 m^2/p.e. Complete nitrification is commonly reported with NH_3-N removal efficiency in excess of 90 percent (Schonerklee et al, 1997, Laber et al, 1999, Cooper, 1999). Typical faecal coliform removal rates in Europe vary from 1.5 to 3 log units (Green et al, 1997, Ottova et al, 1997) while in Nepal, faecal coliform removal rates in excess of 3 log units have been achieved for vertical flow beds (Laber et al, 1997).

2.6.8. Operation and maintenance

The operation and maintenance aspects of VFPGF's are a combination of those of horizontal flow reed beds and infiltration-percolation beds. If beds are to be operated in intermittent mode, with alternating cycles of loading and resting, a reliable operator is required to switch the flow between the beds at the appropriate times.

2.6.9. Advantages and disadvantages

VFPGF's provide enhanced treatment with potential for reuse quality effluent. They are smaller in size than horizontal flow reed beds, and provide better removal of organics and pathogens and full nitrification. Due to their vertical flow mode of operation, their geometry is flexible and they can be made in virtually any shape to fit the available space or blend into the landscape. The disadvantages of VFPGF's are their propensity to clogging at high organic loads and the lack of denitrification capability. They could, however, be used in combination with horizontal flow reed beds to achieve good removal of total nitrogen.