4. RESULTS AND DISCUSSION

4.1. Introduction

This chapter discusses the results of the monitoring and evaluation program of the full-scale systems described in chapter 3 above. Sections 4.2.1 to 4.7 discuss the performance of each of the six treatment systems that were monitored intensively, with sampling of the inlet and outlet of each of the unit processes. Since the systems were designed and implemented over a period of several years, some have been in operation significantly longer than others. The older systems are discussed first, followed by the more recent ones. Section 4.8 discusses the performance of the other treatment systems in each of the different categories, which were monitored for compliance, user satisfaction and reliability. Emphasis, in this section, is given more towards factors, either technical or societal, which caused systems to malfunction or fail, in a practical context, than to the actual quality parameters of the effluents. The effluent was considered as either within compliance, or in violation. Section 4.9 analyses the overall costs associated with the practical implementation of these systems[22], while section 4.10 analyses the costs of the individual unit processes. Section 4.12 evaluates the performance of each of the unit processes across all of the systems.

4.2. The Swiss Residence, System 1
(Ref. Section 3.5.1, page 68)

4.2.1. Plant performance

System 1 of the Swiss Residence has been in continuous operation since the opening of the hotel in December 1997. The system functioned well, resulting in a high level of user satisfaction. In its first year and a half of operation, the effluent of the system was sampled and analysed in order to confirm regulatory compliance, as

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[22] It was found, during the course of this study, that the operation and maintenance costs of the systems were usually negligible in comparison to the cost of construction.

well as safe effluent quality, since the effluent was being reused on-site for gardening and vehicle washing. The sampling frequency was, typically, once a month during this period. The wastewater temperature varied between 25^0C and 28^0C, with occasional drops to 22^0C on rainy days.

The effluent from system 1 was found to be of a consistently high quality, being clear, with no detectable colour or odour. Turbidity was consistently less than 20 NTU, usually in the range of 12 – 16 NTU. Effluent pH was consistently between 6.9 and 7.2. Effluent BOD_5 varied between 9 mg/l and 25 mg/l with a mean of 16 mg/l. Effluent suspended solids varied between 9.6 mg/l and 14.0 mg/l with a mean of 11.7 mg/l. Both these parameters were well within the Sri Lanka standards for discharge of effluents into inland surface waters, which requires 30 mg/l BOD_5 and 50 mg/l suspended solids. Ammonia nitrogen was tested less frequently, but was consistently less than 25 mg/l, which was also well below the required national standard of 50 mg/l. Since the treated effluent was being reused, however, the applicable standards in Sri Lanka were those stipulated for effluents discharged on land for irrigation purposes, which are much less stringent, requiring a BOD_5 concentration less than 250 mg/l and no limit on suspended solids. No requirements exist under this category for ammonia nitrogen or pathogens. Considering the nature of the reuse applications, however, it was deemed appropriate to maintain the World Health Organisation guidelines for the unrestricted reuse of wastewater, which require a faecal coliform count of less than 1000 cfu/100ml and a helminth egg count of less than one egg per litre. Both these requirements were maintained by the effluent, which typically had a faecal coliform concentration in the range of 100 – 500 cfu/100 ml (geometric mean of 238, standard deviation of 158). No helminth eggs were ever detected in the effluent. Figure 4-1 shows a bar chart of the mean, maximum and minimum concentrations for influent and effluent for the system for BOD and suspended solids and their respective treatment efficiencies.

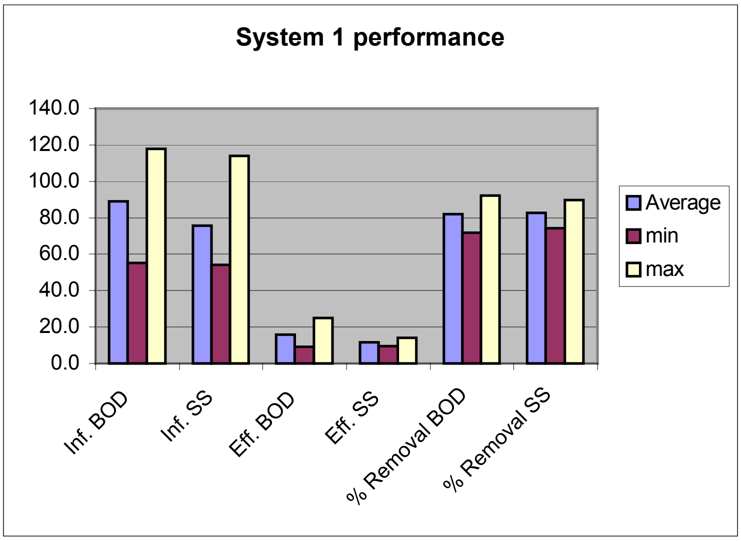

Figure 4-1. Performance characteristics of The Swiss Residence treatment system.

The influent concentrations given are for the influent to the anaerobic filter stage, which is the effluent from the septic tank. Influent to the septic tank could not be sampled due to lack of a suitable sampling point. The removal efficiencies that are given, therefore, do not include the removal efficiency of the septic tank. As can be seen from the bar chart, the influent concentrations were highly variable, with influent BOD_5 varying from 55 to 118 mg/l, with a mean of 88.9 mg/l, and influent suspended solids varying between 54 and 114 mg/l, with a mean of 75.5 mg/l. These values are in general agreement with the effluent characteristics for septic tanks discussed in section 2.2.7 previously. The removal efficiencies for BOD and suspended solids were much the same, with a mean of 82 percent for BOD and 82.5 percent for suspended solids, and a range of 71 to 92 percent for BOD removal and 74 to 90 percent for suspended solids removal. Bearing in mind that these figures exclude the contribution of the septic tank, this would imply a total removal efficiency for the system of around 95 percent including the septic tanks[23]. The wastewater flow to system 1 varied around a mean of 19.7 m^3/d between 11.9 and 23.7 m^3/d. This was within reasonable agreement of the design flow for the system, which was 22.3 m^3/d.

[23] This is by assuming a raw wastewater concentration of around 250 mg/l BOD_5, which is conservative.

Plate 4-1 shows a typical sample of treated effluent from system 1, compared to drinking water from a tap. The tap water sample is on the left and the treated effluent on the right of the picture.

Plate 4-1. A typical effluent sample from system 1 compared to tap water.

4.2.2. Unit process performance

The anaerobic filter and percolation bed units of system 1 were sampled for influent and effluent quality, on a fortnightly basis, for a period of six months from September 1999 to March 2000. Figure 4-2 gives a bar chart of the performance of the anaerobic filter unit for BOD and suspended solids during this period.

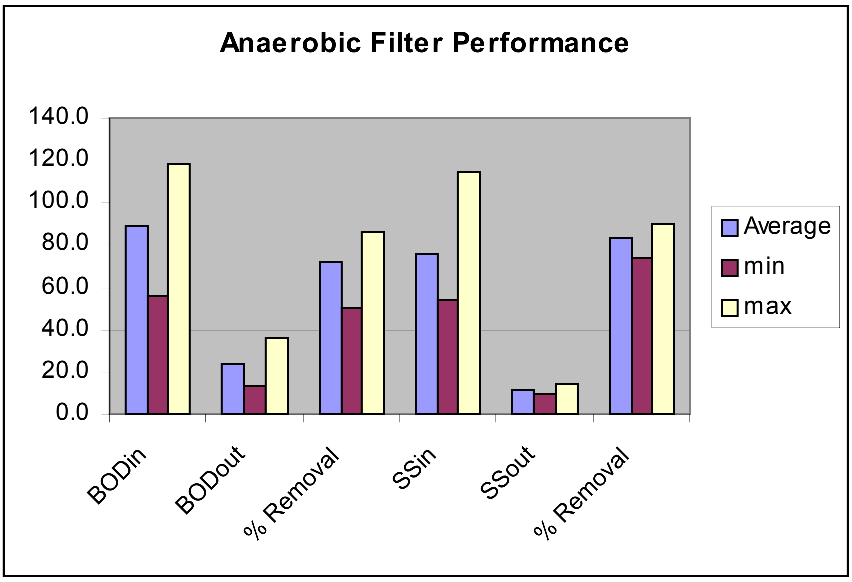

Figure 4-2. Performance characteristics of the Swiss Residence Anaerobic filter.

As can be seen from the figure, the anaerobic filter produced a consistent effluent quality in terms of both, BOD and suspended solids, despite a wide variation in influent quality. The effluent from the unit was almost invariably within the SLS standards for discharge into inland surface waters. The mean effluent concentration was 23.2 mg/l BOD_5 and 11.7 mg/l suspended solids. The range was between 12 and 36 mg/l for BOD, and 9.6 and 14.0 mg/l for suspended solids. The removal efficiency for BOD varied from 50 to 86 percent, with a mean of 72.2 percent. The removal efficiency for suspended solids varied from 74 to 90 percent with a mean of 83.4 percent. The filter did not provide significant removal of turbidity and faecal coliforms, with less than 20 percent removal of turbidity. The pH of the wastewater typically dropped two points between the inlet and outlet of the filter. The influent pH was consistently between 7.0 and 7.3, while the effluent pH was consistently between 6.9 and 7.2.

Figure 4-3 gives the performance characteristics of the percolation bed unit in terms of BOD removal. Since the effluent suspended solids from the anaerobic filter was always below 15 mg/l, no meaningful data could be obtained on suspended solids removal by the percolation bed.

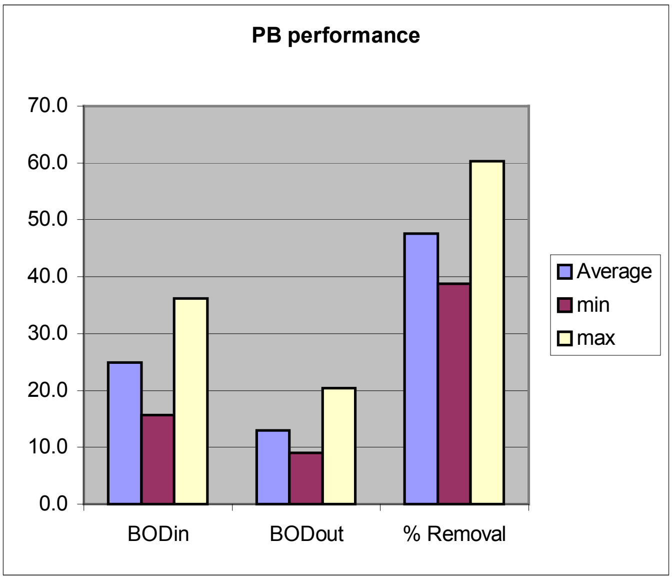

Figure 4-3. BOD_5 removal by the Swiss Residence Percolation bed.

The BOD removal efficiency of the percolation bed unit varied between 39 per cent and 60 percent, with a mean of 47.6 percent. This produced a mean effluent quality of 13 mg/l BOD_5, with a range from 9 to 20 mg/l. Turbidity removal was within a range of 40 to 50 percent, with a mean of 44 percent. There was no significant change in pH across the percolation bed, with the pH remaining consistent between 6.9 and 7.2.

4.2.3. Operation and maintenance

In its first year and a half of operation, system 1 was essentially trouble free, and resulted in a very high level of user satisfaction. When the system was first started up, several leaks appeared in the side of the anaerobic filter unit. These were due to insufficient compaction of the structural concrete during construction and were rectified without much difficulty. On a couple of occasions the system backed up due to gas blockages in the down flow stage of the anaerobic filter. The gases trapped in the filter were released by prodding the filter bed from above, with a stick, which cleared the blockages. This was possible as most of the gas blocks appeared to occur in the upper layers of the filter bed. The gas blocks were found only in the down flow stage of the filter.

During its second year of operation, due to pressure from neighbours and regulators, the hotel management diverted the kitchen effluent as well, to the treatment system. Initially, this did not cause any problems other than an increase in the occurrence of the gas blocks in the anaerobic filter. However, after a few weeks of operation in this mode, the effluent flow rate from the percolation bed decreased gradually, and the system began to back up due to clogging of the percolation bed. The bed was rested for a period of six weeks after which it recovered, partially, but soon clogged again after the flow was restored. Subsequently, the bed was re-laid with a layer of stone chips at the bottom of the filter, with a gravel layer above. When the bed was opened up for relaying, it was found that the clogging had occurred mainly in the gravel around the single effluent collector pipe, which was laid laterally across the bottom of the bed at the outlet end of the bed. Also, it was found that the influent flow was not being distributed evenly over the bed, as the influent distributor pipes had not been laid at the same level during construction. This had resulted in approximately half of the bed being largely unused. A new distributor box was constructed, and the distributor pipes were laid at true level. The effluent collector arrangement was changed to comprise four, parallel, perforated pipes running along the length of the bed at equal spacing. Plate 4-2 shows a plan view of the new arrangement of collector pipes and Plate 4-3 shows the bed being re-laid with a layer of stone chips at the bottom. The original effluent collection arrangement was a single perforated pipe, laid laterally across, at the far end of the bed.

Plate 4-2. The new arrangement of the effluent collector pipes being laid.

Plate 4-3. The percolation bed being re-laid with stone chips.

The septic tank was also desludged at this time, and it was observed that the thickness of the scum layer was far greater than that of the sludge layer in the tank. This was attributed to the grease and oil from the kitchen wastewater. No problems occurred after the rectification work with system 1. It was also decided to convert the down flow stage of the anaerobic filter to up flow mode in order to avoid problems with gas blockage. The filter performance improved dramatically after this was done, and the problem with gas blockages disappeared completely.

4.3. The Swiss Residence, System 2

4.3.1. Plant performance

System 2, which comprised a grease trap, septic tank and VFPGF unit was commissioned in September 1999. The VFPGF unit influent and effluent was sampled fortnightly, for six months, from September 1999 to March 2000.

During the initial start up period, the VFPGF unit was fed with effluent from system 1 in order to allow the plants to establish properly before applying kitchen wastewater to the system. Even after the system was commissioned, however, anaerobic filter effluent from system 1 was continuously pumped to the VFPGF unit, as the percolation bed of system 1 had clogged and rectification work was in progress. This effectively tripled the hydraulic load to the VFPGF unit from a design flow of 7.4 m^3/d to an actual value of 19.7 m^3/d on average, peaking to 24 m^3/d. The system coped reasonably well during this phase in terms of BOD. Figure 4-4 shows the performance characteristics for BOD removal during this phase of operations.

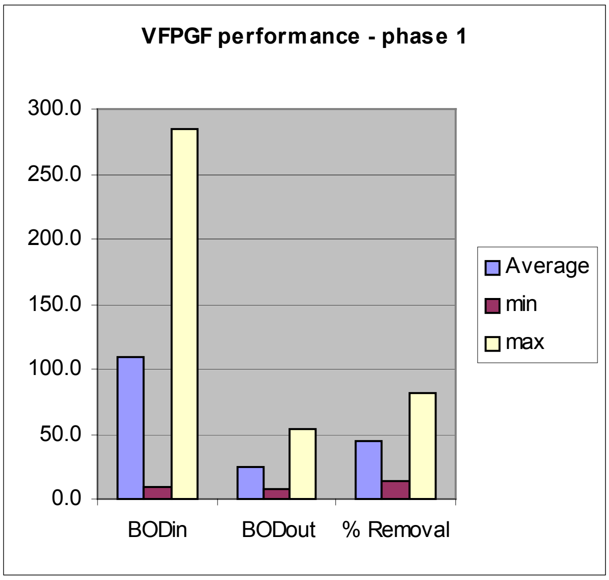

Figure 4-4. BOD_5 removal by the VFPGF unit during phase 1.

As can be seen in the figure, the influent to the unit was highly variable, with a mean BOD_5 of 109 mg/l and a range of 10 mg/l to 285 mg/l. This wide variation in influent concentration was buffered out to a large extent by the system, with a mean effluent BOD_5 of 25 mg/l and a range of 8 mg/l to 55 mg/l. The mean removal efficiency was 44.3 percent with a range of 13.5 to 81 percent. The removal of suspended solids, however, was not consistent, with frequent increase in suspended solids concentration of the effluent in comparison to the influent. This was due to the biomass washing out of the bed during periods of high flow. However, the effluent remained clear and colourless in the main, with no discernible odour. A faint sour odour was detectable within a few metres of the bed during influent dosing, as the distributors sprayed the influent up into the air about half a metre above the bed. The pH was consistently between 6.9 and 7.2 for both influent and effluent.

Once the rectification work of the percolation bed in system 1 was complete, and the system started up once more, System 2 was fed with only kitchen wastewater at an average flow rate of 3.9 m^3/d varying between 2.4 and 6.4 m^3/d, which was within the design flow value of 7.4 m^3/d. However, the pH of the influent to the VFPGF dropped sharply to between 4.2 and 5.0. Figure 4-5 shows the performance characteristics for BOD removal during this phase of operation.

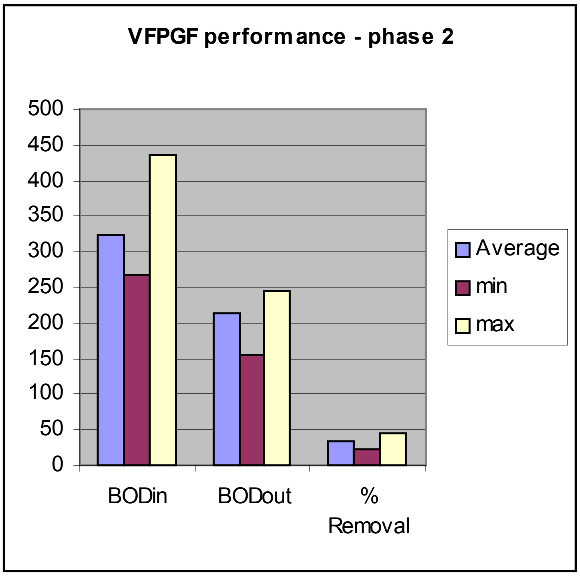

Figure 4-5. BOD_5 removal by the VFPGF unit during phase 2

As can be seen in the figure, the mean influent concentration rose sharply to 324 mg/l BOD_5, and the range was between 266 and 435 mg/l BOD_5. The percentage removal dropped to a mean value of 33 percent and a range of 21 to 44 percent. The effluent was turbid, milky white, in appearance and had a strong sour odour. The effluent pH was elevated by, typically, 5 to 6 points to around 5.6 by the VFPGF unit. However, it was evident that the unit could not cope effectively with the low pH of the septic tank effluent. Figure 4-6 shows the variation of removal efficiency vs. pH for the VFPGF unit, which exhibits a near linear decrease in removal efficiency with decrease in pH below 7.

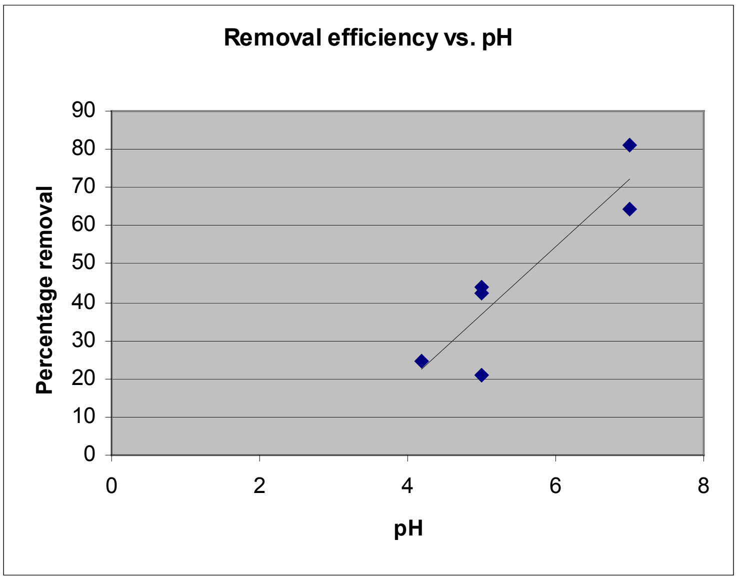

Figure 4-6. Graph of removal efficiency vs. pH for the VFPGF unit.

4.3.2. Operation and maintenance

During phase 1 of operation, the only maintenance activity required was regular cleaning of the foot valve of the dosing pump, which pumped effluent from the kitchen septic tank to the VFPGF unit. The foot valve was protected from solids by a plastic mosquito mesh wrapped around the unit. However, this mesh would regularly get clogged with biomass growth and require cleaning.

During the second phase of operation, the orifices in the influent distributor to the VFPGF would occasionally get clogged with grease and require cleaning. At first, a steel wire was inserted into the orifices in order to clear the clogging. Subsequently, removable end caps were installed at the end of each lateral. By removing each end cap, sequentially, during influent dosing, the accumulated grease and biomass would wash out of the end of the lateral under the pressure of the dosing pump. An attempt was made to elevate the pH of the influent to the VFPGF by dosing the influent sump with lime. However, the pH was heavily buffered between 4 and 5 and this was largely unsuccessful. Within six months of operation in phase 2, the bed was showing signs of clogging, with influent ponding on the surface of the bed during dosing. The clogging worsened gradually at first, and then, more rapidly with the connection of the laundry wastewater to the system in November 2000.

The original grease trap was supposed to be cleaned monthly. However, the hotel maintenance staff did not attend to this, as the accumulated scum and grease in the trap had a strong foul odour. This resulted in large amounts of oils and grease escaping into the system, which further contributed to the failure of the system. Consequently, it was decided to install a smaller, daily-cleaned, grease trap instead. Since this trap could be cleaned easily, on a daily basis, the scum and grease did not have a chance to decompose and generate odours. Also, an anaerobic filter of 1.5 days HRT was built between the septic tank and VFPGF to further treat the septic tank effluent before being applied to the VFPGF. However, by this time, the VFPGF had clogged almost completely, and had to be taken out of service in order to rest and restore the unit. Meanwhile, the effluent from the anaerobic filter that was built for system 2 was fed to the anaerobic filter of System 1. This arrangement has been working well for a period of three months, with the anaerobic filter and percolation bed of system 1 coping well with the additional load, without signs of stress or a drop in the overall effluent quality of system 1. Figure 4-7 shows the current configuration of the two treatment systems.

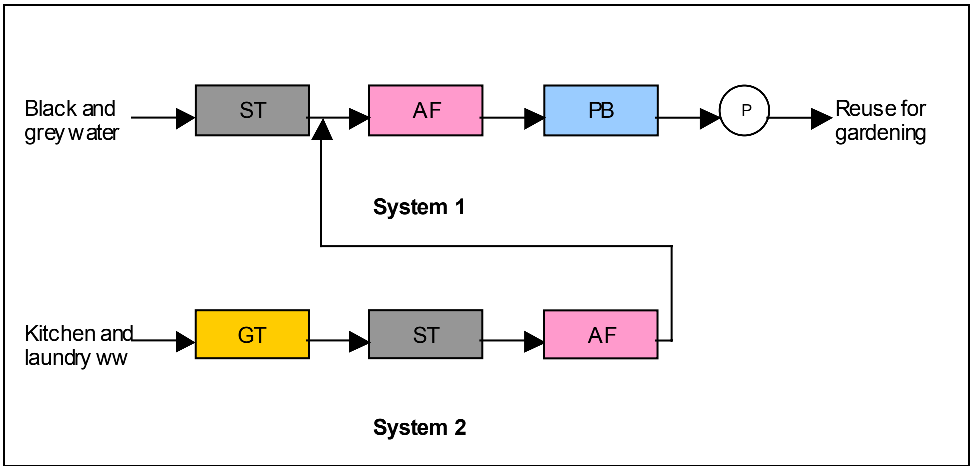

Figure 4-7. Current configuration of Swiss Residence treatment systems.

4.4. Akbar - Nell Hall.

(Ref. Section 3.8.2, page 122)

4.4.1. Plant performance

The Akbar - Nell treatment system was sampled on a fortnightly basis for two months in May and June of 1999 and then again for six months from September 1999 to March 2000. Samples were taken from the inlets and outlets of each of the two inner channels in each of the two stages of the beds. The outer channels of the beds in both stages were not operated due to leaks.

From the outset, it was evident that the design flow value of 25.5 m^3/d grossly underestimated the actual wastewater flow from the hall, which was found to average over 123 m^3/d. This was mainly due to the poor maintenance of the toilet fixtures in the building, in which the taps and cisterns were running continuously, in many cases. Also, no allowance had been made for the large number of non-resident students who used the hall cafeteria at lunchtime. From the point of view of the reed beds, this problem was compounded by the fact that 50 percent of the bed area, i.e.

the two outer channels of the beds, could not be utilized due to leaks arising from construction defects, which had not been rectified. Figure 4-8 shows the basic flow characteristics through the reed bed system.

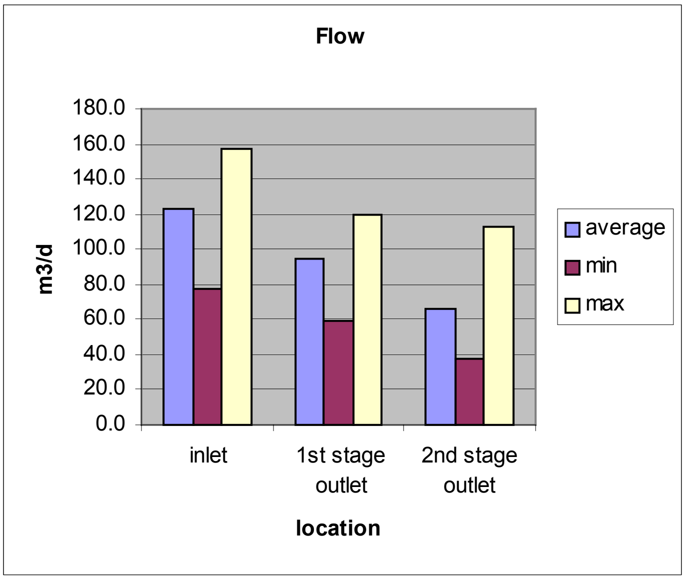

Figure 4-8. Wastewater flow through the reed bed system.

The flow to the inlet of the first stage ranged between 77.5 m^3/d and 158 m^3/d, with a mean of 123.2 m^3/d. This flow reduced to a mean of 94.6 m^3/d and a range of 59 to 120 m^3/d by the outlet of the first stage. This reduction was mainly due to seepage through the bottom of the bed, due to shoddy construction, rather than evapo-transpiration by the plants, as the reduction in flow was present, virtually unchanged, when the beds were not vegetated. The flow in the second stage of the filter reduced even further, dropping to an average of 65.5 m^3/d and a range of 38 m^3/d to 113 m^3/d at the outlet. Figure 4-9 shows the individual flow characteristics of the two channels that were in operation.

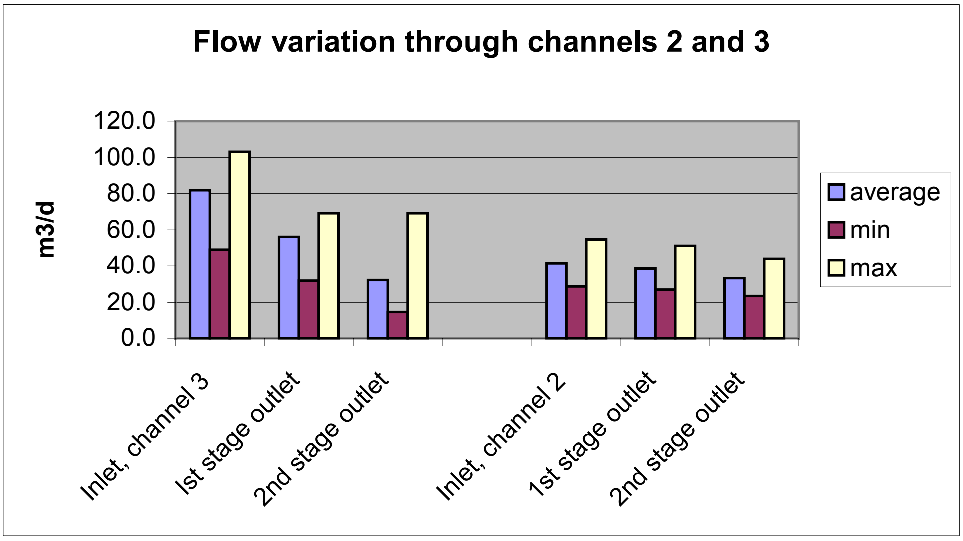

Figure 4-9. Flow characteristics of the individual reed bed channels

As can be seen in the figure, the influent flow was unevenly divided between the two channels. Channel 3 received an average flow of 81.5 m^3/ d at the inlet, with a range of 48 to 103 m^3/ d. Channel 2 received almost fifty percent less flow, with an average of 41.3 m^3/ d and a range of 29 to 54 m^3/ d. The flow loss through the bed was also greater in channel 3 which experienced an average loss of 60 percent of the inflow, reducing the average effluent flow from the 2nd stage outlet to a mean value of 32.1 m^3/ d, and a range of 14 to 69 m^3/ d. The flow loss in channel two was less, with an average loss of 19 percent of the flow from the inlet to the outlet of the second stage.

The increase in actual flow over the design value, together with the unequal flow distribution, effectively increased the hydraulic and organic loading of the beds by several hundred percent.
Table 4-1 shows the design and actual values of hydraulic and organic loading rates as well as the resulting reduction in specific area for each of the four beds.

Table 4-1. Comparison of design and actual loading rates for the Akbar- Nell reed beds

	1st stage		2nd stage	
	Channel 2	Channel 3	Channel 2	Channel 3
Design HLR (m/d)	0.26	0.26	0.26	0.26
Actual HLR (m/d)	1.65	3.26	1.54	2.24
Hydraulic overload factor	6.4	12.5	5.9	8.6
Design OLR ($gBOD/m^2.d$)	38.1	38.1	6.9	6.9
Actual OLR ($gBOD/m^2.d$)	81.3	135	31.5	46.6
Organic overload factor	2.1	3.5	4.6	6.8
Design specific area (m^2/p.e.)	0.625	0.625	0.625	0.625
Actual specific area (m^2/p.e.)	0.10	0.05	0.10	0.07

The 1st stage bed of channel 3 has experienced the highest hydraulic loading rate with an average of 12.5 times the design value, while the 2nd stage bed of channel 3 has experienced the highest organic overload, with 6.8 times the design organic loading rate. In actual terms, however, the 1st stage bed of channel 3has experienced the highest organic loading rate with 135 $gBOD/m^2.d$. Figure 4-10, Figure 4-11, and Figure 4-12 present a graphical representation of these results.

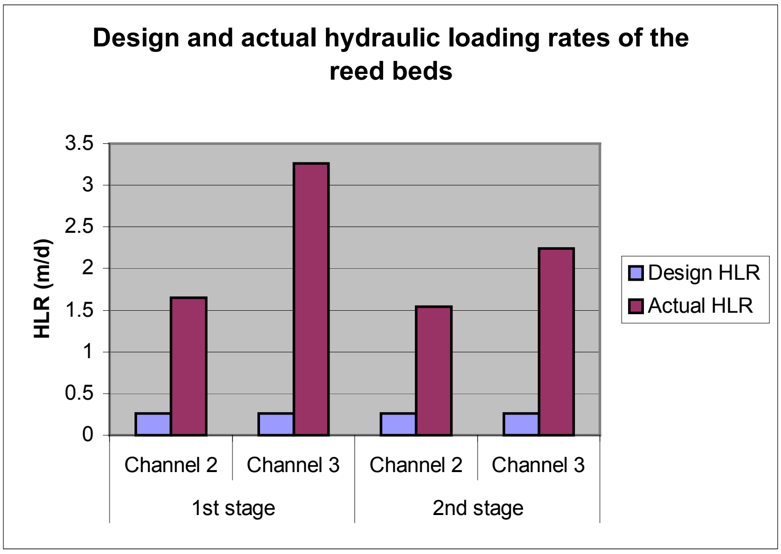

Figure 4-10. Comparison of design and actual hydraulic loading rates

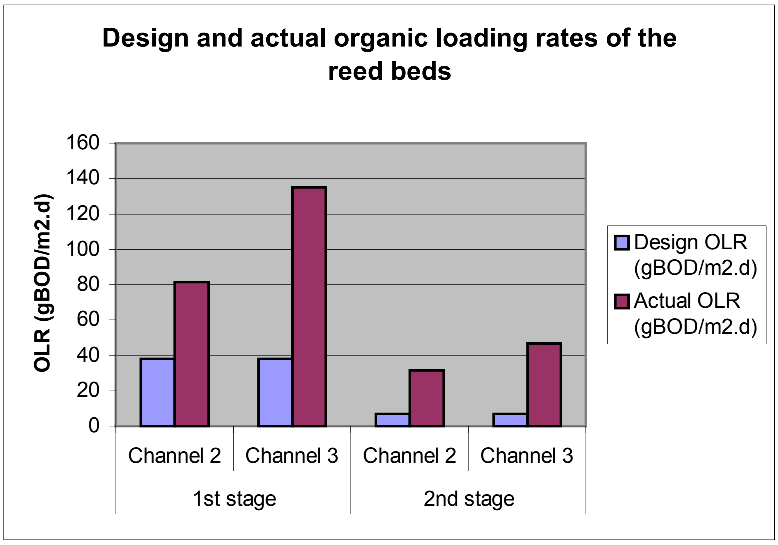

Figure 4-11. Comparison of design and actual organic loading rates.

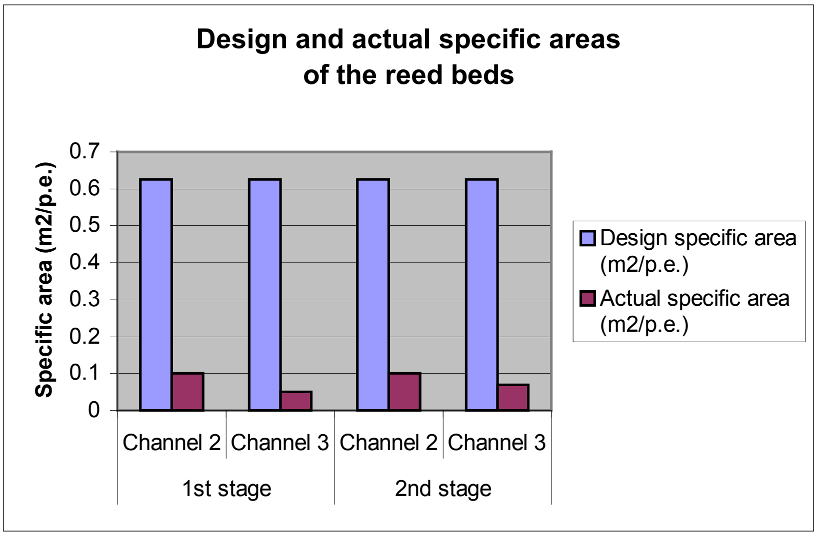

Figure 4-12. Comparison of design and actual specific areas of the reed beds.

Despite the overloading of the reed beds, they performed well in terms of overall effluent treatment, and produced a consistent, clear effluent with no detectable colour or odour. Figure 4-13 and Figure 4-14 show the variation of influent and effluent BOD through the two stages of each of the reed bed channels over the same four-month period.

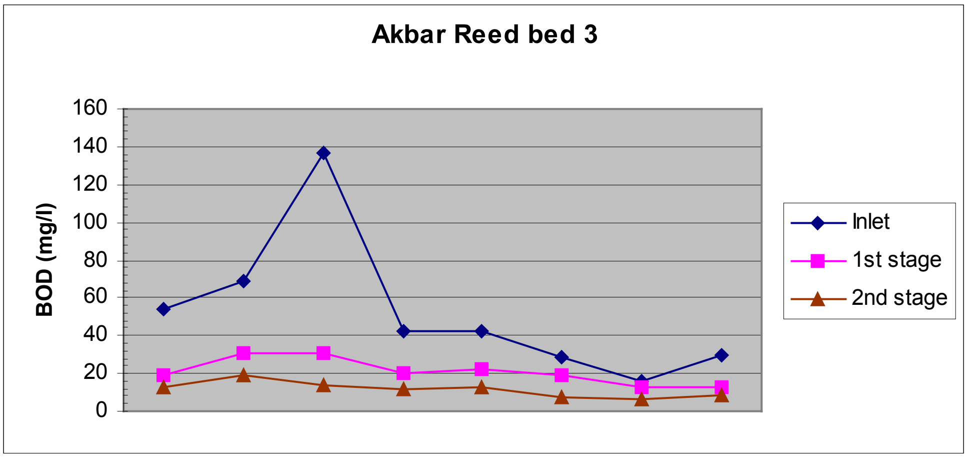

Figure 4-13. Variation of BOD_5 concentration in Channel 3 over a four-month period.

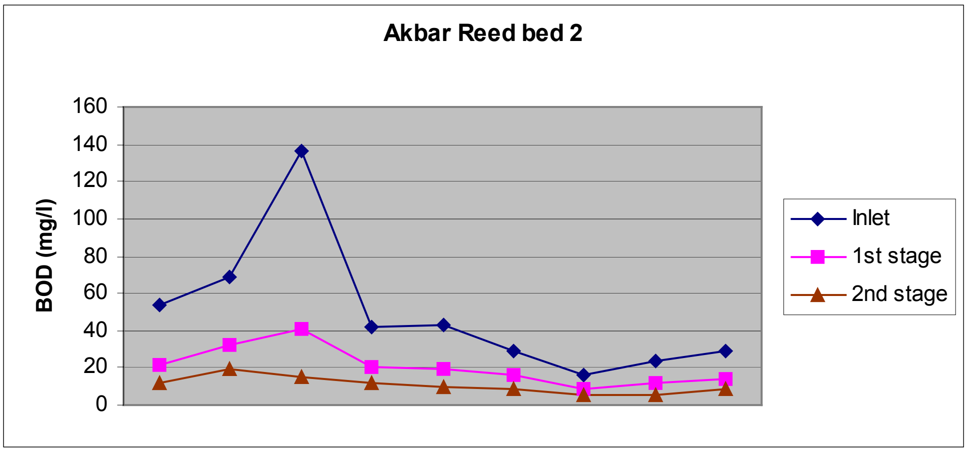

Figure 4-14. Variation of BOD_5 concentration in Channel 2 over a four-month period.

As can be seen from the graphs, both the channels performed well in producing an effluent quality consistently below the discharge standard of 30 mg/l. In fact, channel 3 was consistently within discharge standards by the outlet of the 1st stage. Channel 2, though less heavily loaded, hydraulically, was not that consistent, with the effluent from the 1st stage going up to 40 mg/l BOD_5 on occasion. It should be mentioned here, that only the first half of the two 1st stage beds were vegetated during the period of intensive sampling, while the entire lengths of the second stage beds were fully vegetated. This was coincidence rather than a planned occurrence. Figure 4-15 shows the overall performance of the reed beds as a system in terms of BOD removal.

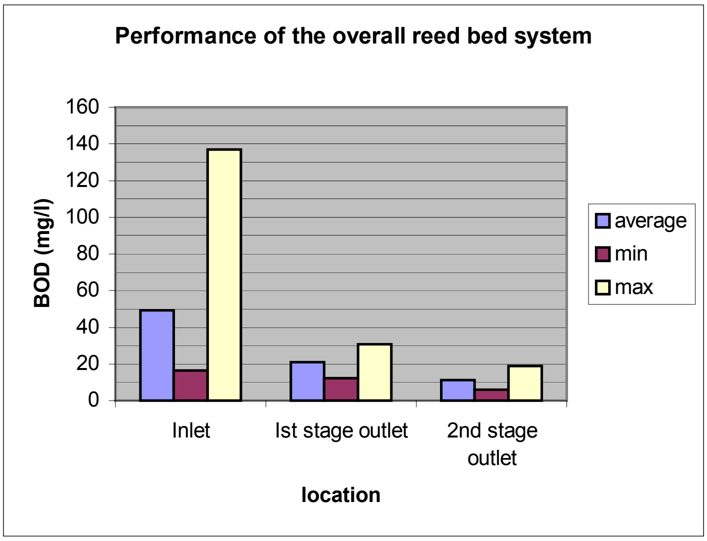

Figure 4-15. Overall performance of the system in terms of BOD removal

As can be seen from the figure, the system was effective in damping out wide variations in influent concentration to produce a consistent, high quality effluent, despite heavy overloading. The overall influent concentrations varied from 16 to 137 mg/l, with a mean of 49 mg/l. The average influent concentration to the 1st stage beds was much lower than the design value of 150 mg/l. This was due to dilution of the raw sewage with clean water, from the poorly maintained taps and cisterns in the hall. The final effluent concentrations varied between 6 and 19 mg/l with a mean value of 11 mg/l. The effluent from the 1st stage beds varied between 12 and 31 mg/l with a mean of 21 mg/l, which on average was slightly lower than the design influent concentration for the second stage beds, which was 27 mg/l. The mean final effluent concentration was also slightly lower than the design value of 15 mg/l. Figure 4-16 shows the performance of the system in terms of BOD removal efficiency.

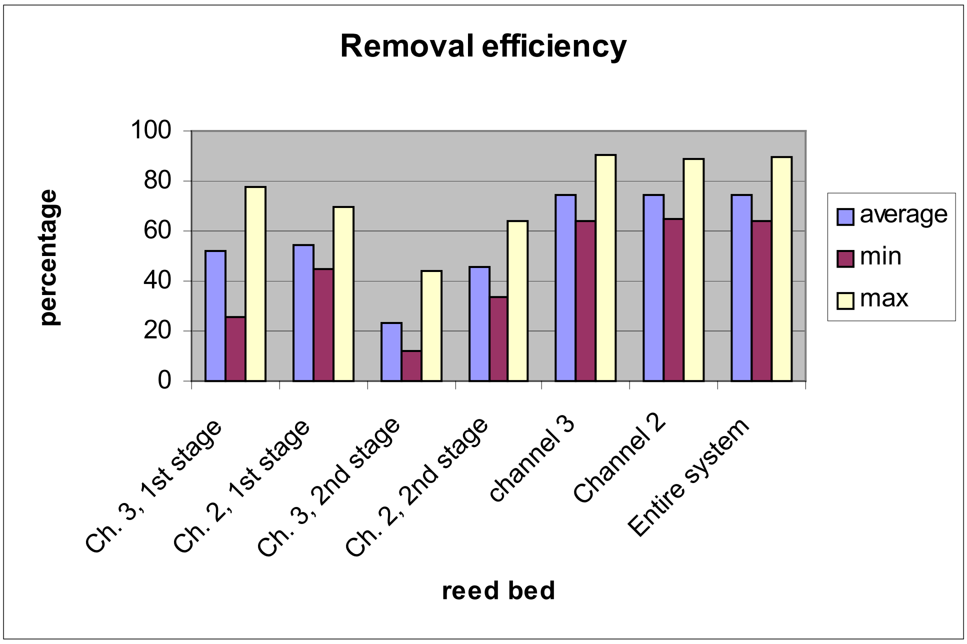

Figure 4-16. BOD removal efficiencies for the reed bed system

As can be seen in the figure, the channel 3 beds showed a wider range of removal efficiency in comparison to channel 2. The 1st stage bed of channel 3 had a mean removal efficiency of 52 percent, with a range of 26 to 78 percent, while the 1st stage bed of channel 2 had a mean removal efficiency of 54 percent, with a range of 45 to 70 percent. In the second stage, channel 3 had a mean removal efficiency of 23 percent, with a range of 12 to 44 percent, while channel 2 had a mean of 46 percent, with a range of 23 to 64 percent. However, the overall efficiencies of each channel, when considering both the stages, as well as the overall efficiency of the entire system as a whole, were very similar, with channel 3 having a mean of 75 percent, with a range of 64 to 90 percent, and channel 2 having a mean also of 75 percent, with a range of 65 to 90 percent. The overall system had the same figures.

Figure 4-17 shows the inlet and outlet concentrations of suspended solids of the individual reed beds as well as for the system as a whole.

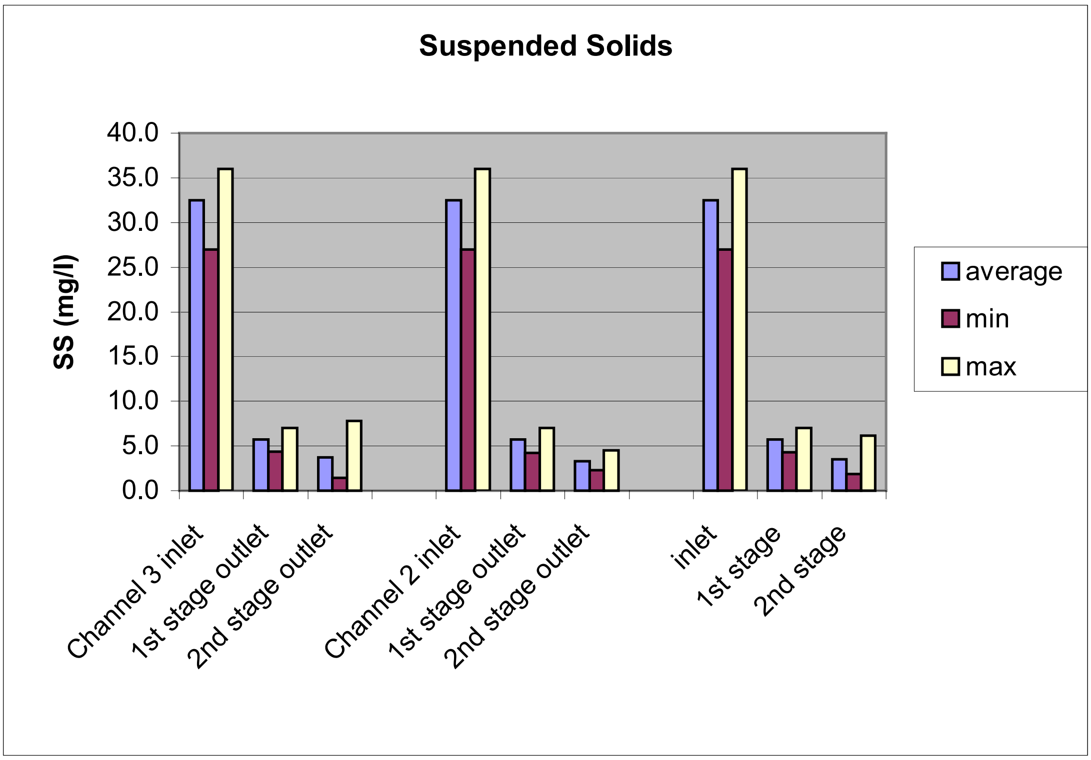

Figure 4-17. Performance of the reed beds in terms of Suspended Solids.

As can be seen in the figure, the effluent quality of all the beds was excellent in terms of suspended solids. Here, too, the influent concentrations were slightly lower than the anticipated values, due to dilution of the raw sewage with water from the leaking toilet fixtures. The mean influent concentration was 32.5 mg/l SS, with a range of 27.0 to 36.0.

Turbidity declined steadily through the reed beds, as can be seen in Figure 4-18, which shows the variation of turbidity through the reed bed stages.

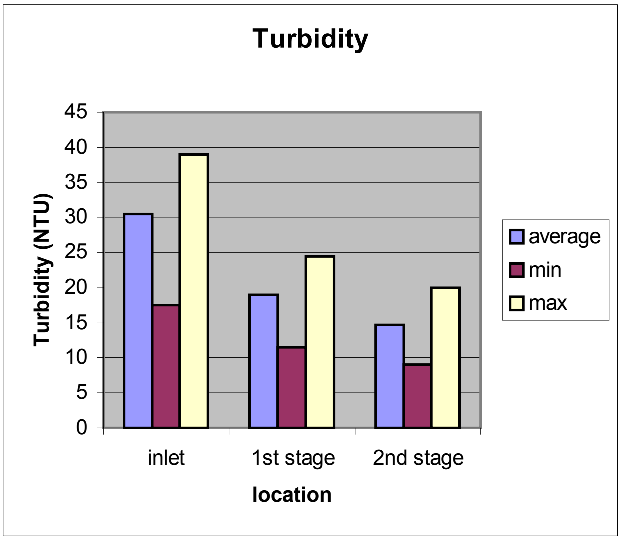

Figure 4-18. Variation of turbidity through the reed bed stages.

Turbidity at the inlet varied between 18 and 39 NTU, with a mean value of 31 NTU. The outlet turbidity of the 1st stage varied between 12 and 25 NTU, with a mean of 19 NTU, while the turbidity of the final effluent varied between 9 and 20 NTU, with a mean of 15 NTU.

Ammonia Nitrogen removal performance of the reed beds could not be assessed effectively, because the influent concentrations were too low, typically varying between 2 and 8 mg/l. This was probably due to the effect of dilution of the raw sewage as discussed previously. Helminth eggs, too, were present in the influent only sporadically, so their removal by the beds could not be properly assessed. No eggs were found in the effluent of any of the beds. Faecal coliform concentrations were typically in the order of 10^3-10^4 cfu/100 ml in the influent to the reed beds (geometric mean 4538, standard deviation 3093), with, typically, an order of magnitude removal in each of the stages, giving a final effluent concentration between 10 - 100 cfu/100 ml (geometric mean 49.2, standard deviation 38.3). However, the faecal coliform data at hand was insufficient to make a definitive assessment of the removals in each stage.

4.4.2. Operation and maintenance

The operation and maintenance of the Akbar-Nell treatment system was, officially, the responsibility of the University maintenance division, which was the same organization responsible for the maintenance work of the other Hall facilities as well. The approach of the maintenance division was to ignore the system unless some sort of crisis was brought to their notice. Once the system was built, the sewage was connected, without the beds being vegetated. This condition prevailed for several months. An effluent channel was not built to convey the final effluent to the stream, which was the intended discharge point. Instead, the effluent was left to meander along the ground and finally flow along a gravel road passing by the reed bed system. This was compounded by the fact that the two outer channels of the reed beds had severe leaks due to construction defects. The flow when applied to these channels would not arrive at the outlet point. Instead it would leak out from under the beds and flow along the gravel road. The maintenance reaction was to shut down the two outer channels. However, the inner channels too, had leaks though not as severe, which is evident from the flow data presented in Figure 4-9 of section 4.4.1 above. This eventually resulted in the passers by who used the road on a regular basis, taking matters into their own hands and shutting the control valve to the reed beds. This further compounded the problem by causing, the raw influent to overflow from the flow distribution box and meander along the ground, causing severe problems of odour in the vicinity. Meanwhile, untended vegetation growing wild around the beds, invaded the outer channels, covering them up completely. In essence, the institution concerned demonstrated a consistent lack of will and ability to perform simple maintenance activities, such as repairing of leaks, cutting an effluent channel, maintenance of vegetation etc. Plate 4-4 shows the gravel road beside the treatment system which became completely saturated and waterlogged with leaking sewage from the reed beds.

Plate 4-4. The gravel road beside the system waterlogged with leaking sewage from the beds.

The unvegetated beds, though, produced a reasonably good quality effluent. However, with time, a biomat was observed to form on the surface of the bed at the inlet end, which caused local clogging in that area of the bed. This biomat steadily progressed down the bed, until the entire bed was covered and completely clogged. Plate 4-5 shows a view of an unvegetated channel with partial clogging of the bed. The biomat is visible on the surface of the bed in the lower half of the picture. Plate 4-6 shows a view of a reed bed channel, which had become completely clogged over its entire length. Water was stagnant over the bed surface and was highly active with mosquito and other insect larvae. It was observed that this clogging was a surface phenomenon, and the stagnant water could be released by prodding the surface of the bed with a stick. Plate 4-7 shows a view of the surface biomat after the water had been released in this manner.

Plate 4-5. An unvegetated reed bed channel with inlet zone clogging.

Plate 4-6. A completely clogged reed bed channel.

Plate 4-7. The biomat layer soon after releasing the stagnant water.

Eventually, out of desperation, the beds were vegetated on a voluntary basis, by some of the students and the two inner channels brought into proper operational condition. The vegetation was established by cutting stems of common reeds from nearby reed stands, and sticking them into the bed at approximately 30 - 60 cm intervals. The surface clogging was cleared by prodding the bed surface at regular intervals with a stick, but the biomat itself was allowed to remain. The reeds quickly established themselves within a couple of weeks and the biomat gradually disappeared completely, with the establishment of the vegetation. Plate 4-8 shows the planted reed stems in the bed a couple of days after planting. The biomat is visible in the upper part of the channel. Plate 4-9 shows a close up view of the bed with the vegetation fully established and the biomat completely cleared. This condition prevailed as long as the beds remained vegetated.

Plate 4-8. The reed stems a couple of days after planting

Plate 4-9. A close up view of the fully vegetated bed with no sign of biomat formation.

The present condition of the system is that passers by have completely shut down the system once more, due to the inconvenience caused to them by the waterlogging of the road due to the leaks in the beds. As a result, the beds have become, for all intents and purposes, abandoned, with untreated effluent overflowing from the distribution box and flowing overland to the stream nearby. Severe problems of odour and mosquitoes prevail in the vicinity due to the stagnant pools of effluent. Plate 4-10 shows an example of the poor state of repair of the system, with outlet flow control pipes broken and in a poor state of repair.

Plate 4-10. The effluent discharge elbows in a poor state of repair.

4.5. Ladyhill Hall

(Ref. Section 3.8.1, page 120)

4.5.1. Plant performance

The Ladyhill system was one of the earliest systems to be implemented, and has been in continuous operation since early 1996. The influent and effluent of the anaerobic filter were sampled on a fortnightly basis for a period of almost 18 months from mid 1996 until the end of 1997, and then again for one month in March 2000. The system was always working close to design capacity, in terms of flow. Although the flow itself could not be measured directly, occupancy of the building was monitored and the flow was estimated. The average flow estimated by this method was 1.96 m^3/d, and the range was 1.12 - 6.0 m^3/d. The latter value is three times the design flow value of 2 m^3/d, which was experienced during a single 24-hour period. No significant change in effluent quality was experienced as a result of the temporary overload. The effluent quality was consistent, clear and devoid of any detectable colour or odour. Figure 4-19 shows the variation of influent and effluent quality over the long-term sampling period.

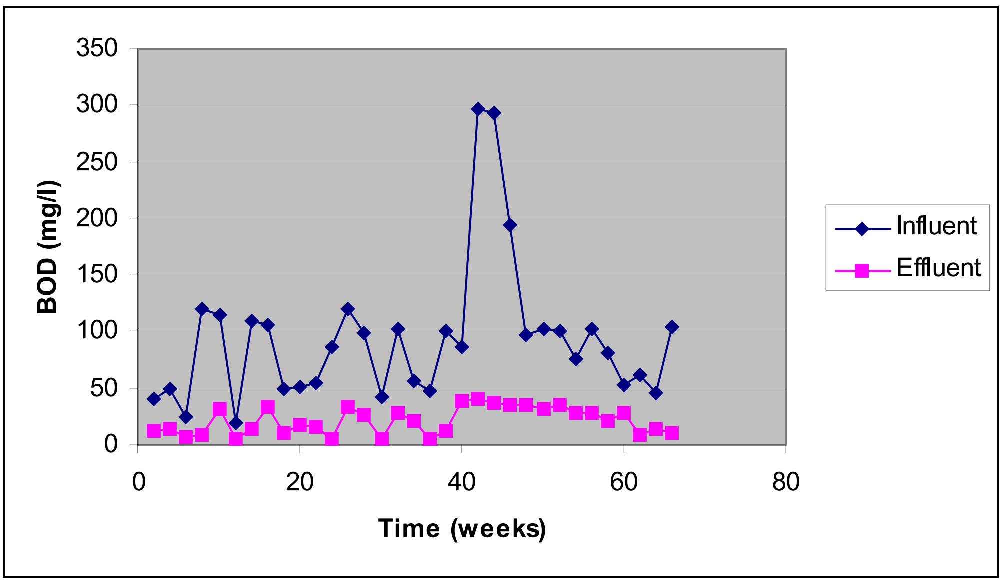

Figure 4-19. Long-term performance of the Ladyhill anaerobic filter

As can be seen in the figure, the filter was capable of buffering out the effects of both, organic and hydraulic shock loads. The influent BOD_5 concentration varied between 20 and 290 mg/l, with a mean of 93 mg/l. The effluent concentration varied between 5 and 40 mg/l with a mean of 23 mg/l. The anaerobic filter was inadvertently dosed with insecticide on one occasion, which resulted in most of the higher effluent BOD values between 30 and 40 mg/l. Some problems with odour also occurred during this period.

The suspended solids concentration in the influent to the filter was always low, with a mean value of 23 mg/l and a range of 16 to 34 mg/l. This was probably due to the large triple chamber septic tank, which preceded the filter. The effluent suspended solids concentration varied between 2 and 12 mg/l with a mean value of 8.4 mg/l. Figure 4-20 shows a graphical representation of these values.

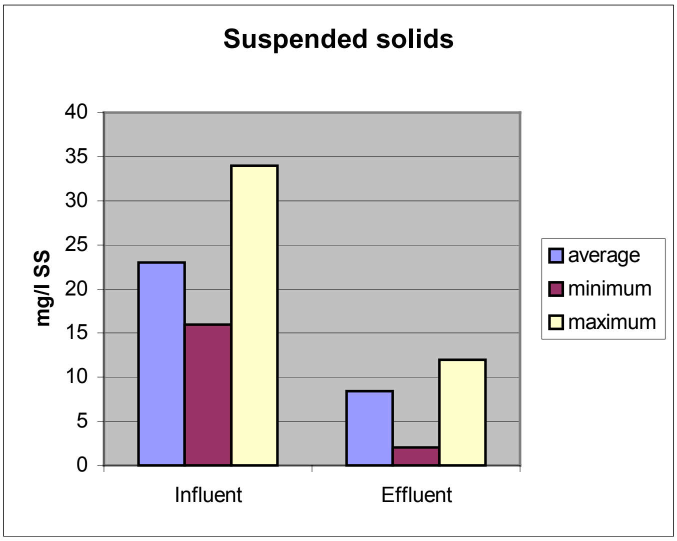

Figure 4-20. Suspended solids concentrations for the Ladyhill anaerobic filter

Ammonia nitrogen was not monitored at an intensive level. However, the effluent concentrations were checked occasionally for regulatory compliance, and ammonia nitrogen levels were typically in the range of 15 - 30 mg/l with no violations of the 50 mg/l discharge standard during the entire period of operation. Faecal coliforms were not removed to a significant extent by the filter, with a single order of magnitude reduction between influent and effluent at best. No helminth eggs were detected either in the influent, or the effluent of the filter.

4.5.2. Operation and maintenance

The operation and maintenance of the Ladyhill treatment system was also the responsibility of the same University maintenance division, which was responsible for Akbar-Nell system. However, since the anaerobic filter required no regular maintenance activity once commissioned, the system has been functioning well without any problems. Mosquitoes were found to be breeding inside the anaerobic filter unit, which was easily stopped by covering the vent pipe of the filter with plastic mosquito mesh. The unit has been functioning without any form of complaints, either from the occupants, or regulators, except for minor complaints of odour on two occasions from occupants on the ground floor immediately overlooking the backyard where the system is buried. The first was due to odour from the filter and septic tank unit due to improper sealing of the manhole covers. Once attended to, the problem subsided together with the complaints. The second occasion was the one referred to earlier, when the filter had been dosed with insecticide. The problem subsided of its own accord within a couple of days.

4.6. Devon Rest

(Ref. Section 3.5.3, page76)

4.6.1. Plant performance

The Devon Rest treatment units were sampled for influent and effluent quality for a period of six months from December 1999 to May 2000, on a fortnightly basis. Since then, the Kandy Municipal Council has monitored the effluent quality monthly, for compliance. Overall, the performance of the system has been good, in terms of effluent quality and user satisfaction. Figure 4-21 shows the BOD values for the anaerobic filter influent, anaerobic filter effluent, and percolation bed effluent, during the period December 1999 to May 2000.

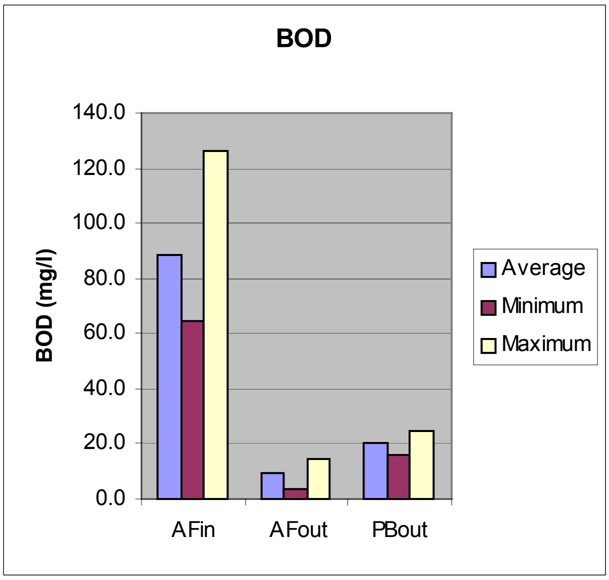

Figure 4-21. BOD_5 values for the Devon Rest treatment system.

As can be seen in the figure, the anaerobic filter has been performing well in terms of BOD removal. The influent BOD varied between 64 and 126 mg/l, with a mean of 88.5 mg/l. The effluent BOD from the anaerobic filter varied between 3.9 and 14.4 mg/l with a mean of 9.1 mg/l, which is very good quality. Interestingly, this quality appeared to deteriorate in the percolation bed, which had a mean effluent BOD of 20.5 mg/l, and a range of 16 to 25 mg/l. The reason for this apparent deterioration in quality was that the original septic tank, which was an old brick structure, was leaking septage through the soil, into the percolation bed. However, despite this leaking of septage, the percolation bed effluent was well within discharge limits. The suspended solids values for the system are presented in Figure 4-22.

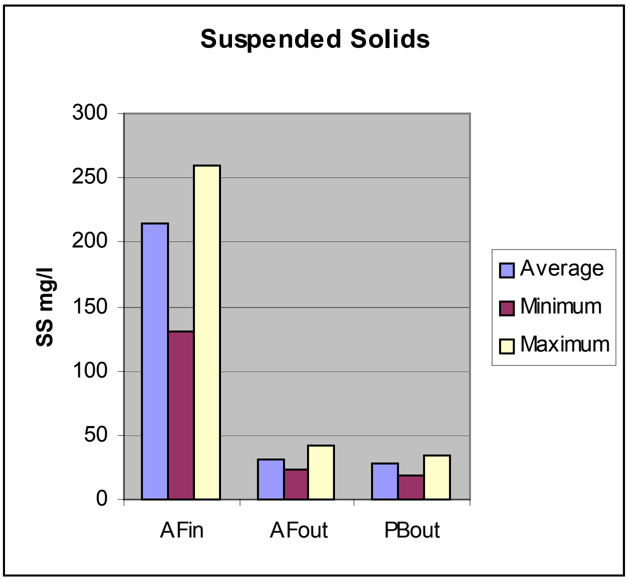

Figure 4-22. Suspended Solids values for the Devon Rest treatment system

As can be seen in the figure, the anaerobic filter performed well in terms of suspended solids as well. The influent suspended solids concentrations to the filter were highly variable, as may have been expected, with a mean value of 215 mg/l and a range of 130 to 260 mg/l. This was dealt with very effectively by the filter, which produced a mean effluent concentration of 31.5 mg/l, and a range of 23.5 to 42.0 mg/l. The effluent suspended solids from the percolation bed had a mean concentration of 28.5 mg/l, with a range of 19 to 33.5 mg/l. The removal efficiencies in terms of BOD and suspended solids for the anaerobic filter are shown in Figure 4-23. Meaningful removal efficiencies for the percolation bed cannot be evaluated due to the problem of septage leaking into the bed.

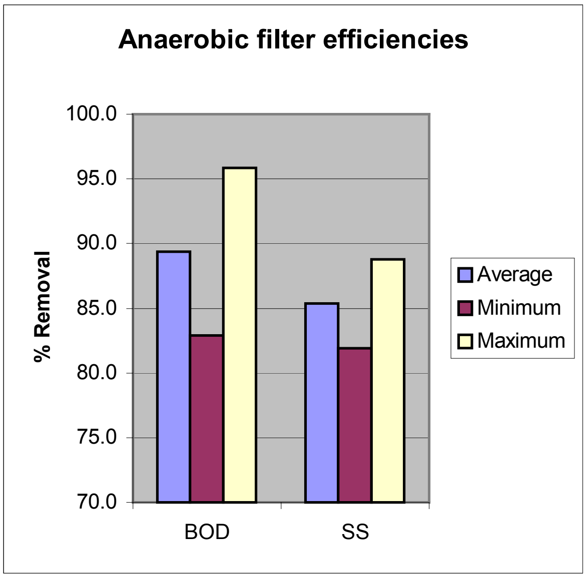

Figure 4-23. Removal efficiencies of the anaerobic filter unit

As shown in the figure, the mean removal efficiency for BOD in the anaerobic filter was 89.4 percent, with a range of 83 to 96 percent. The mean removal efficiency for suspended solids was 85.3 percent, with a range of 82 to 89 percent.

The pH of the influent to the anaerobic filter varied between 5.5 and 6.4, with a mean of 6.1, while the effluent from the anaerobic filter varied between 6.0 and 6.8, with a mean of 6.4. Typically, the pH was observed to increase by 4 to 5 points from inlet to outlet of the filter. The pH did not vary significantly across the percolation bed. Turbidity removal in the filter was not significant and typically reduced by 10 percent across the filter from between 40 - 45 NTU at the inlet, to between 35 and 40 NTU at the filter outlet. The percolation bed was more effective in turbidity removal, with a near consistent value of 25 NTU in the effluent with very little variation. Ammonia nitrogen, however, was found to increase steadily with time in the percolation bed effluent, increasing from 30 mg/l to 48 mg/l over a four-month period. This was also probably due to the leaking of septage. The gradual increase with time could be explained by accumulation of ammonia nitrogen in the septage and surrounding soil due to anaerobic decomposition of organic nitrogen, which then leaked into the percolation bed.

4.6.2. Operation and maintenance.

The Devon Rest system has been in operation for approximately one and a half years since September 1999. The maintenance activity has been limited to cleaning the grease trap of the kitchen wastewater line on a regular basis. This is a daily-cleaned grease trap. Some problems with odours from the septic tanks occurred due to improper sealing of the access manholes. The problem disappeared once the manhole covers were sealed. After a year and half of operation, the anaerobic filter started overflowing on one occasion. The cause was found to be an air block in the pipeline from the anaerobic filter to the percolation bed. Other than these occasional minor events, the system has been functioning well, with a high level of user satisfaction. The problem of the leaking of septage from the septic tank was discovered during excavation of the pit for the percolation bed unit. It was noticed at the time that black, foul smelling, water was seeping through a particular soil stratum intersected by the excavation. This problem stopped after repairing cracks in the lining of the septic tank. However, it appears to have recurred again, as the sampling results indicate. One reason is that the original septic tank had been built, resting on shallow bedrock, using the bedrock as its bottom without an additional base. However, the management of the hotel is reluctant to remedy the situation, as they see no benefit in doing it as the system is functioning to their satisfaction as well as that of the regulators. Plate 4-11 shows the excavation for the percolation bed with the black horizontal band of soil saturated with leaking septage visible in the centre of the cut on the left and far sides of the excavation. The leaking septic tank is behind the rubble masonry retaining wall visible on the upper left corner of the picture.

Plate 4-11. The excavation for the percolation bed intersecting the black soil layer.

4.7. Kadugannawa

(Ref. Section 3.6.1, page 104)

4.7.1. Plant performance

The Kadugannawa treatment system was sampled on a fortnightly basis for five months from March to July 2000. The occupancy during this time was an average of 2- 3 persons, with frequent peaks of 7 - 10 persons during weekends. The design occupancy was 5 persons. The system performed well, overall, with a high level of user satisfaction. The effluent from the reed bed as well as the VFPGF unit was always clear, with no detectable colour. A faint odour was detectable from the effluent pipe of the reed bed when it was discharging high flows. This was discernible only within a metre or so of the outlet pipe of the reed bed. The effluent from the VFPGF unit had no discernible odour at any time. Figure 4-24 shows the BOD values for the reed bed and VFPGF units.

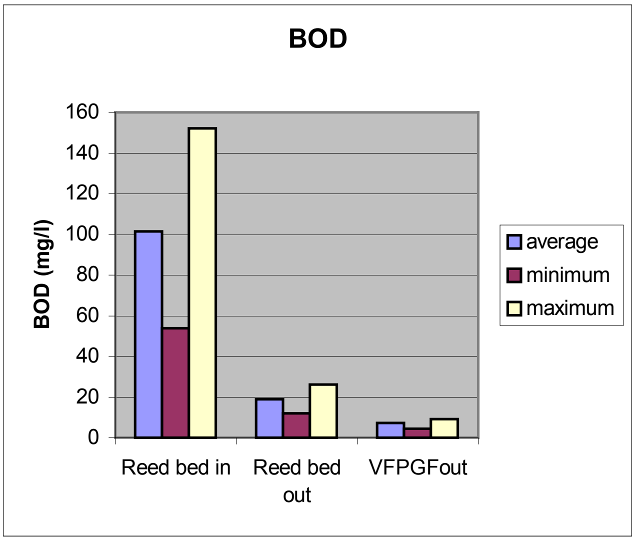

Figure 4-24. BOD_5 performance of the Kadugannawa treatment system

As can be seen in the figure, the influent BOD to the system was highly variable, ranging between 54 and 152 mg/l, with a mean value of 102 mg/l. The effluent from the reed bed was consistently between 12 and 26 mg/l, with a mean of 18.8 mg/l, while the effluent from the VFPGF varied between 4.5 and 9.2 mg/l with a mean of 7.4 mg/l. Figure 4-25 shows the suspended solids values for the units.

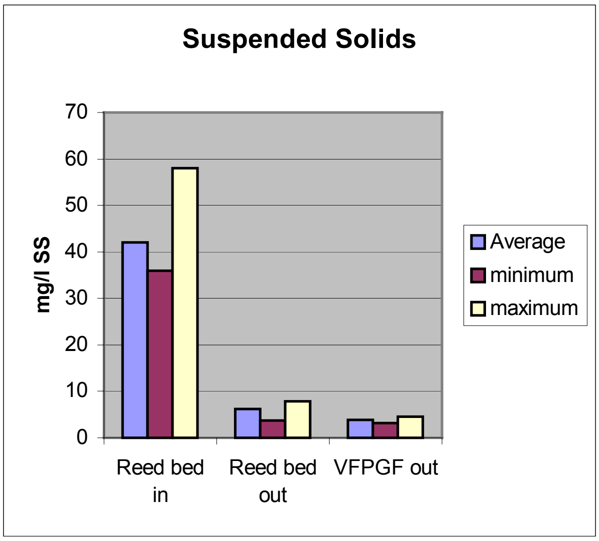

Figure 4-25. Suspended solids performance of the Kadugannawa system

As can be seen in the figure, the suspended solids concentration in the influent to the reed bed varied between 36 and 58 mg/l, with a mean value of 42 mg/l. The reed bed effluent varied between 4.1 and 7.8 mg/l with a mean of 6.2 and the VFPGF effluent varied between 3.2 and 4.6 mg/l, with a mean of 3.9. Figure 4-26 shows the respective removal efficiencies for the units.

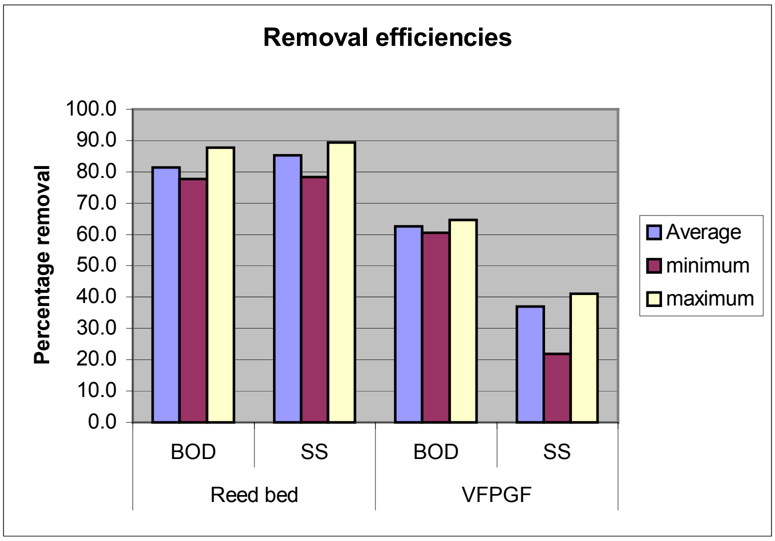

Figure 4-26. Removal efficiencies of the Kadugannawa treatment units

As can be seen in the figure, the removal efficiency for both BOD and suspended solids was around 80 percent for the reed bed. The actual values are a mean of 81.5 percent, and a range of 77.8 to 87.6 percent for BOD, and a mean value of 85.2 percent, with a range of 78.3 to 89.3 percent for suspended solids. The corresponding removal efficiencies were lower for the VFPGF unit, with a mean value of 62.5 percent, with a range of 60.6 to 64.6 percent for BOD removal, and a mean value of 37.1 percent, with a range of 22 to 41 percent for suspended solids removal. It should be noted here that the influent to the VFPGF unit was already of a very high quality in terms of both BOD and suspended solids, which would probably account for the lower removal efficiencies.

The performance was equally good in terms of faecal coliforms, with a reduction of two orders of magnitude in the reed bed, and a further two orders of magnitude reduction in the VFPGF unit. The range of values for faecal coliforms was 10^5 - 10^6 cfu/100ml in the influent to the reed bed (geometric mean 485,000, standard deviation 244,000), down to 10^3 - 10^4 cfu/100ml in the reed bed effluent (geometric mean 3982, standard deviation 3534) and 60 - 350 cfu/100ml in the final VFPGF effluent (geometric mean 170, standard deviation 126). Helminth eggs were not detected in the influent or the effluent of any of the units. Ammonia nitrogen concentrations varied between 10 and 25 mg/l in the reed bed influent, down to between 8 and 16 mg/l in the reed bed effluent. The levels in the VFPGF effluent were between 2 and 6 mg/l.

4.7.2. Operation and maintenance

The regular operation and maintenance activity for the Kadugannawa system, which has been in operation since November 1999, was limited to caring for the vegetation in the beds and occasional cleaning of the distributor pipes and orifices of the VFPGF. These would get blocked mainly due to leaves and other external debris falling into the open outlet basin of the reed bed and subsequently entering the distributor pipes with the wastewater flow. This could have been avoided very simply by covering the outlet basin. However, it was not perceived to be a serious enough problem by the occupants to necessitate preventive action. The reed bed was originally vegetated with common reeds, similar to those in the Akbar-Nell beds. However, the occupants gradually replaced the reeds with ornamental plants such as *Cannas* spp. and *Coleus* spp. This change in vegetation had no significant effect on the performance of the bed in terms of treatment. However, the broad-leafed plants caused a marked drop in the effluent flow from the bed, due to losses through evapo-transpiration. This effect was more pronounced in the dry months.

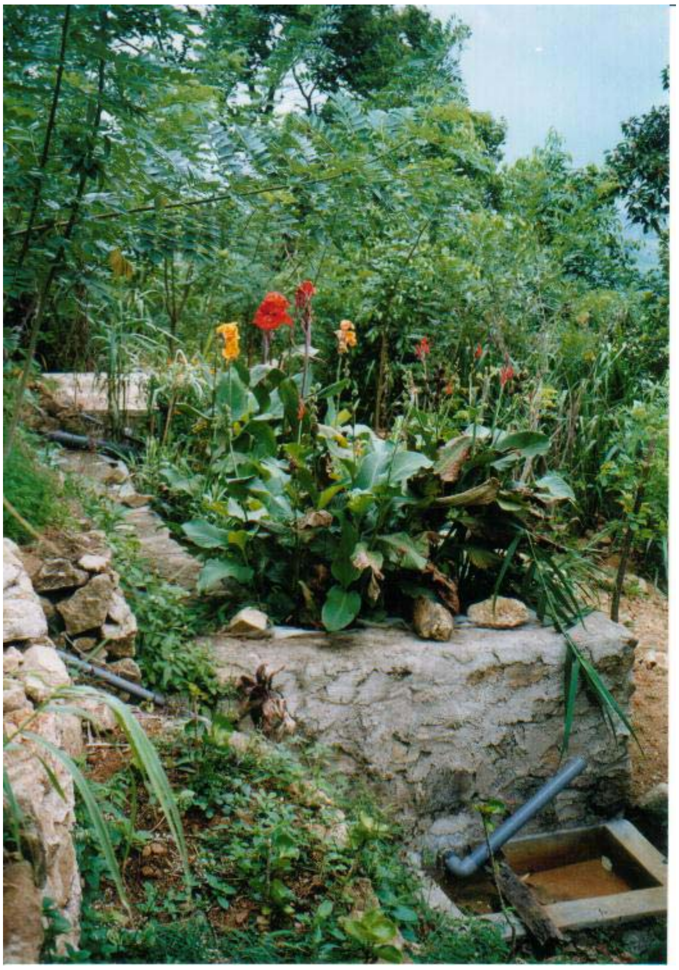

Plate 4-12. The Kadugannawa reed bed with the open outlet tray

Plate 4-12 shows a view of the reed bed unit with the outlet level control pipe and outlet tray visible in the foreground. Soil and leaf litter would fall in to the open tray and get carried in to the VFPGF distributor pipes

The VFPGF unit was originally designed and operated as an open percolation bed. However, the occupants decided to vegetate the bed for ornamental purposes. This did not cause a significant change in the performance of the unit in terms of treatment. It was often observed that the influent was not evenly distributed over the bed by the distributor pipes, and only a small fraction of the bed area was being actually utilized (approximately 20 - 30 percent). This was mainly because the VFPGF was gravity fed, at low head, rather than the influent being pumped intermittently as in the other VFPGF units in operation. Also the PVC distributor pipes tended to warp in the sunlight and cause preferential flow through some of the orifices.

4.8. Other Treatment systems

4.8.1. Hotels

The other hotel treatment systems were Devon Hotel, Hotel Thilanka (Systems 1 and 2), Kings Park, Ivy Banks, Coral Sands and Wattles Inn. In general, all these treatment systems have performed well and, invariably, better than the Swiss Residence systems and the Devon Rest system discussed previously. This was mainly because these systems were designed and implemented later and benefited from the experience of the previous two.

Devon hotel system (Ref. Section 3.5.2, page 73) has been in operation since May 1999, and produced a consistent, high quality effluent, which has been always clear, and devoid of colour and odour. A few complaints arose from an immediate neighbour soon after the system was commissioned regarding odour in the common storm water drain into which the system discharged, which was resolved by extending the effluent discharge pipe to discharge directly into a nearby stream. This made it evident that the odour in the drain, which still persisted, was not caused by the effluent, but by other discharges of untreated grey water and dumping of garbage etc. into the drain further upstream. Both users and regulators have been satisfied with the performance of the system, which has always been within regulatory compliance since it's commissioning in early 1999. The only maintenance activity has been the cleaning of the grease trap and the foot valve of the effluent pump of the kitchen septic tank. The foot valve gets stuck regularly with growth of biomass on the valve, as well as with small particles getting trapped in the valve, which has required regular cleaning.

System 1 of Hotel Thilanka (Ref. Section 3.5.4, page 79) has been in operation since August 2000, and has been in operation for seven months. The performance has been excellent, with a very high quality effluent and very high levels of user and regulator satisfaction. Plate 4-13 shows a typical sample of effluent from the Thilanka treatment system. The effluent is virtually indistinguishable from drinking water in terms of both, visual and olfactory quality.

Plate 4-13. A typical effluent sample from the Thilanka treatment system.

The VFPGF of system 1 was commissioned in stages, with the influent loading being increased incrementally, with sequential connecting of different parts of the hotel to the system. Upon initial commissioning, a strong, earthy odour emanated from the VFPGF unit during influent dosing cycles, although the effluent itself was devoid of any detectable odour. The odour decreased steadily as the system matured and disappeared completely within 2 - 3 weeks. This phenomenon kept repeating, to a somewhat lesser extent, each time a new component of inflow was connected to the system, with the odour disappearing within a few days after the incremental load. The same phenomenon was experienced in the VFPGF units of System 2 as well, and appeared to be a characteristic of these units.

System 2 was also commissioned in stages, beginning in September 2000. This system, too, produced a very high quality effluent. However, some operational problems occurred due to a flock of wild monkeys, which visit that part of the hotel premises regularly on a daily basis. The monkeys destroyed the vegetation in the VFPGF units, and tampered with the valves controlling the influent dosing to the units, which interfered with their operation. It was finally decided to enclose all the VFPGF units in cages of welded mesh to keep the monkeys out. Also, with the connecting of the final stage of the system, which was effluent directly from a septic tank receiving wastewater from the staff kitchen, office block and drivers and guides quarters, the VFPGF unit began to show signs of clogging. The clogging first occurred in the second stage units, which had a higher hydraulic loading rate than the first stage units, and manifested itself approximately a month after connection of the septic tank effluent. This occurred simultaneously with the monkeys destroying the vegetation in the units, which compounded the problem. Wastewater began to pond on the surface of the beds, which began to cause problems of odour. The problem was further compounded by soil washing into the dosing siphons of the first stage VFPGF due to torrential rains, which blocked some of the distributor pipes of the unit. There was a marked drop in effluent quality during this period, with the effluent becoming turbid, with some amount of odour present. The clogged second stage beds were cleaned and the vegetation re-established in both the stages. The dosing siphons were protected from surface water and the distributor laterals in the VFPGF's equipped with end caps that could be conveniently unscrewed for cleaning. The system has been functioning well since then, for a period of two months, and the effluent quality restored.

The Kings Park treatment system (Ref. Section 3.5.6, page 91) was commissioned in October 2000, and has been functioning well for a period of four months, with a high level of user and regulator satisfaction. This was a complete turnaround for a hotel, which previously had been targeted by regulators, environmental watchdog organizations and concerned public for discharging untreated effluent into the Kandy Lake and threatened with imminent closure. The effluent has been of consistently high quality, with typical concentrations of 18 mg/l BOD_5, 15 mg/l SS, and 15 mg/l ammonia nitrogen[24]. Similar problems to that of Devon hotel were experienced with the foot valves of the pumps. However, in this case, it had more serious

[24] Based on regulatory monitoring data from the Kandy Municipal Council laboratory.

consequences as the sump was under the front lawn of the hotel, which needed to be dug up to access the foot valve. This problem was over come by devising a manhole cover with a steel edge around it. The edging protruded a few centimetres above the manhole cover, and supported a soil layer on top of the cover, which was then turfed. This turfed manhole cover fitted neatly into the existing lawn without compromising its appearance, and could be lifted off for access, turf and all, when necessary.

The Ivy Banks treatment systems (ref. Section 3.5.5, page 88) were commissioned in October 2000, and have been functioning for four months. Some initial problems were experienced with clogging of the percolation bed, which directly received all the grey water other than kitchen wastewater. Subsequently, a small, combined grease trap and grit chamber was built prior to the percolation bed, which appears to have solved the problem.

The Coral Sands treatment system (Ref. Section 3.5.7, page 95) was commissioned in November 2000, and has been functioning for four months. The performance has been excellent, particularly in comparison to System 2 of the Swiss Residence, which is the only other system, which exclusively handles kitchen wastewater from a hotel. The anaerobic filter unit is functioning well. The VFPGF bed incorporated a layer of limestone to elevate the pH and reduce odour, which has been very effective. The unit was located within less than a metre of windows of staff rooms. No complaints have been made regarding odours, except for a complaint of mild odour during hot sunshine hours in the afternoons. This was probably due to volatilisation of gases in the bed due to heating up of the bed, and was overcome by shading the bed and increasing the vegetation cover. The Wattles Inn treatment system (Ref. Section 3.5.9, page 100), though designed, was not implemented because of the planned expansion of the hotel being abandoned due to problems of financing.

4.8.2. Housing Schemes

The Ranpokunugama systems 1 and 2 (Ref. Section 3.7.1, page 112) have been in operation for a period of over two years since January of 1999. The systems were built under the supervision of the National Housing Development Authority, through tendered contractors. Once built, they were handed over to the local Pradeshiya Sabha (village council) for operation and maintenance. The anaerobic filter units of system 1 have been working well throughout the period, with a high quality effluent. The presence of the units, which are buried in the yard of a house in the scheme, and discharge to a stream at the bottom of the property, have been since 'forgotten' by the occupants of the house, who previously, had been very conscious of the failing soakage pits, which the units replaced. The top slab of the filter units, which was flush with the ground surface was considered a regular part of the yard and utilized for activities such as stacking of firewood, drying laundry etc. No maintenance activity has been required or carried out since the units were commissioned.

The reed bed units of system 2 were also claimed to be functioning well from the point of view of the local Pradeshiya Sabha and the neighbouring residents. However, the vegetation in the beds gradually died out soon after commissioning, due to the fact that the outlet level control elbows had been fixed in a permanently 'empty' position as far as the beds were concerned without possibility of varying the water depth in the bed. Consequently most of the flow in the beds was along the bottom of the bed and the plants could not establish themselves in the early stages due to lack of water near the bed surface. Also, due to lack of maintenance of the surrounding vegetation, plants had invaded the inlet flow distribution channel of the beds resulting in most of the flow being diverted through a single channel of the four-channel system. By August 1999, i.e. after approximately eight months of operation, this channel showed early signs of inlet zone clogging. This clogging was very similar in nature to that of the Akbar-Nell reed beds, though less severe, and was mainly due to formation of a biomat on the surface of the bed. The effluent, though, was always clear and free of odour, which led to the high level of user and neighbour satisfaction. The NHDA was, however, persuaded to rectify the situation by installing the proper outlet level control elbows, which would enable adjusting of the water levels to the appropriate height, as well as to cover the inlet flow distribution channel to prevent invasion and clogging by external vegetation. The long-term maintenance of the system, though, as far as maintaining vegetation, is uncertain, judging by the level of maintenance activity of the local Pradeshiya Sabha. Plate 4-14 shows the condition of the reed bed system after the first eight months of operation. Most of the flow had been along the bottom of the third channel from the left, which was showing early signs of clogging as is evident from the surface flow visible on the near part of the channel.

Plate 4-14. The Ranpokunugama reed beds after eight months of operation

Plate 4-15 shows a close up view of the clogged area of the reed bed channel. This clogging could be easily cleared by prodding the surface of the bed with a stick, to puncture the biomat and drain the bed surface. However, unless vegetation was properly established, the clogging would recur.

Plate 4-15. A close up view of the clogged area of the reed bed.

Plate 4-16 shows the clogged area immediately after the surface flow had been drained as described earlier. If vegetation were to be established together with the proper flow level in the bed at this stage, the clogging would disappear.

Plate 4-16. A view of the channel immediately after clearing the clogged area

Both the Luisawatte systems have been built. However, they are not operational as yet, as the housing units in the scheme have not come into occupation as yet. The Ceylinco systems are still under construction as is the Poorwarama system.

4.8.3. Houses

The Talwatte system (Ref. Section 3.6.5, page 109) has been in operation for over five years, since late 1995. The system has been functioning well with a high level of user satisfaction. No maintenance activity of any kind has been required or carried out for the system. In fact, the owner has become accustomed to the maintenance-free operation of the system, and is usually surprised when contacted to inquire about the system, that an inquiry is being made at all.

The Moratuwa and Kelaniya house systems (Ref. Section 3.6.6, page 110 and Section 3.6.4, page 109, respectively) have been operational for two years and have been free of any maintenance activity in the main. Some initial complaints were made regarding the effluent stagnating in the roadside drain in front of the house at Kelaniya. However, these subsided once the drain was cleared and levelled appropriately to prevent stagnation of water. The Nugegoda system (Ref. Section 3.6.2, page 107) has been built, but the house remains, as yet, unoccupied. The Nawala system (Ref. Section 3.6.3, page 108) was built and commissioned in mid 2000. However, due to problems in construction, particularly laying of the HDPE liner of the VFPGF unit, the wastewater leaks out through the bottom of the bed and percolates into the ground, resulting in the effluent being unavailable for reuse according to the original objective of the system.

4.8.4. Schools and Halls of Residence
(Ref. Sections 3.8.3, and 3.8.4, page125)

The Jayathileke system was built and commissioned, without vegetating the reed beds, in mid 2000 by the University maintenance department. It appears to be functioning satisfactorily for the moment, but in the long-term, probably awaits a fate similar to that of the Akbar-Nell reed bed system. The Kal Eliya system, though designed, has yet not been built, due to problems of financing experienced by the school.

4.8.5. Day-time occupancy buildings
(Ref. Section 3.9, page 127)

The Avanhala treatment system was commissioned in September 2000 and has been operational for five months. The system has been functioning well, with no maintenance activity required or complaints recorded.

The Sampath Hall system was commissioned in June 2000, and has been operational for eight months. The system has been trouble free, in the main, and no maintenance activity has been required. The Kandy Municipal Council laboratory, which monitors the effluent for regulatory compliance, reports typical effluent quality to be in the region of 18 mg/l BOD_5, 20 mg/l SS and 30 mg/l ammonia nitrogen. However, a few complaints of odour from the effluent discharge point have been recorded on days of large functions. This happens when the Hall has remained unused for a continued period of time and suddenly hosts a large function. The anaerobic filter probably becomes dormant during extended periods of negligible flow, and is less able to cope with the sudden influx of a large flow.

The Seeduwa warehouse complex was commissioned in February of 1999, and has been operational for two years. The system appears to be functioning well, with no complaints of any sort, and no maintenance activity required.

The PGIA system has been in operation for over four years, and the system has been working well with no complaints of any sort and no maintenance activity required. The Engineering library complex has been in operation for two years, and this too has been working well with no maintenance activity required, and no complaints. The Ceylon Cold Stores System is still under construction.

4.9. System Costs

4.9.1. Hotels

Table 4-2 gives the actual implementation costs for the different hotel systems in terms of year 2000 SLR's. The corresponding conversion rates for US Dollars and Pound sterling are 117 SLR's per GBP and 76 SLR's per USD respectively. The population equivalent of the flow handled by the system as well as the treatment process and method of effluent disposal are also provided in summary form. In this case, reuse refers to reuse for gardening, landscaping and vehicle washing, and discharge implies discharge to surface drains or streams. The last column shows the specific cost of the systems in terms of cost per p.e. The two Thilanka systems have been considered as comprising two subsystems each, in order to better reflect the system cost. The subsystems only share common VFPGF units within each system, and the costs of these have been apportioned proportionally.

Table 4-2. Implementation costs of hotel treatment systems

System	p.e.	Process	Effluent disposal	Implementation Cost (SLR)[25]	Cost / p.e. (SLR/p.e.)
Swiss residence 1	140	ST-AF-PB	Reuse	1,057,000	7550
Swiss residence 2	46	ST-VF	Discharge	170,000	3696
Devon Hotel	108	ST-AF-PB	Discharge	2,000,000	18519
Devon rest	48	ST-AF-PB	Discharge	392,000	8167
Thilanka 1a	170	ST-VF	Reuse	350,000	2059
Thilanka 1b	142	ST-AF-VF	Reuse	1,170,000	8239
Thilanka 2a	90	ST-AF-VF	Reuse	680,000	7556
Thilanka 2b	86	ST-AF-VF	Reuse	600,000	6977
Ivy banks	28	ST-AF; PB	Soakage	100,000	3571
Kings Park	69	ST-AF-PB-VF	Discharge	865,000	12536
Coral Sands	19	ST-AF-VF	Discharge	432,000	22737
Wattles Inn	108	ST-AF-VF	Reuse	1,070,000	9907

Table 4-2 shows a wide range of specific cost for the hotel systems ranging from a minimum of 2059 SLR's per p.e, to a maximum of 22,737 SLR's per p.e, which is over ten times the minimum. The average system cost per p.e is 9293 SLR's per p.e. The wide range of unit costs is not surprising, given that some of the systems utilized existing septic tanks as primary treatment units, the costs of which are not reflected in the system costs, while other systems included the costs of new septic tanks which were built as part of the system. Therefore, the comparison of unit costs in this form is not particularly productive. Furthermore, the costs shown in the table are the implementation costs of the system in its original design configuration. It does not include the cost of rectification of design failures such as in the Swiss residence

[25] Costs given are in Year 2000 SLR's with 76 SLR's per USD and 117 SLR's per GBP.

system 2 which did not perform satisfactorily in its original configuration, and subsequently had an anaerobic filter added together with being combined with System 1, in order to achieve satisfactory performance.

Table 4-3 compares the costs of the systems, altered to reflect the cost had they all had septic tanks built as part of the systems, and the corresponding costs had all of them utilized pre-existing tanks for primary treatment. The hypothetical cost of the tanks was calculated on a bill of quantities based on an actual design of the tanks for each case, and rated according to the actual construction rates encountered for each project. The data for Swiss Residence system 2 have been altered to reflect the cost of the functioning system in its final form, which is a combination of the original systems 1 and 2 together with the addition of an anaerobic filter unit. The numbers given in italics are values adjusted in the manner described above using hypothetical costing of septic tanks.

Table 4-3. Implementation costs of hotels adjusted for purposes of comparison

System	p.e.	With septic tanks		Without septic tanks	
		Cost (SLR)[26]	Cost/p.e. (SLR's/p.e)	Cost (SLR)	Cost/p.e. (SLR's/p.e)
Swiss 1	140	1057000	7550	*636000*	4543
Swiss 2	186	1327000	7134	*801000*	4306
Devon hotel	108	2000000	18519	*1158000*	10722
Devon rest	48	*447000*	9313	*325000*	6771
Thilanka 1a	170	*877400*	5161	350000	2059
Thilanka 1b	142	1170000	8239	*840000*	5915
Thilanka 2a	90	*1163000*	12922	680000	7556

[26] Costs given are in Year 2000 SLR's with 76 SLR's per USD and 117 SLR's per GBP.

Thilanka 2b	86	*889000*	10337	600000	6977
Ivy banks	28	100000	3571	*72000*	2571
Kings Park	69	*1002000*	14522	*799000*	11580
Coral sands	19	432000	22737	*303000*	15947
Wattles Inn	108	1070000	9907	733000	6787

A few of the systems appear to have a disproportionately high specific cost compared to the others, even after adjusting for the septic tanks. In particular, Coral Sands has a very high specific cost of 22737 SLR/p.e, as do Devon hotel and Kings' Park with 18519, and 14522 SLR/p.e. respectively. Both Coral Sands and Devon Hotel were built under difficult conditions for construction, as well as the units were designed to withstand heavy vehicle loads due to their locations. Coral Sands was built in loose, sandy soil with a high groundwater table, necessitating extensive shoring to support excavation as well as continuous dewatering during construction. The same was true of Devon Hotel. Although the soil wasn't as loose as in the case of Coral sands, the excavation was much deeper and continuous dewatering was required, together with shoring of excavation. Also, Devon hotel had a separate septic tank for kitchen wastewater buried under the basement, which required pumping of effluent to the main system. The necessity for a separate kitchen septic tank drove up the cost in comparison to handling all the sewage in one, larger unit. This point is validated by the specific cost value considered without the septic tanks, which is much closer, in comparison, to that of the other systems. The Coral Sands system was designed to handle only kitchen wastewater, which is difficult to treat on its own, as was experienced in the Swiss Residence system 2. This necessitated larger units and greater safety in design.

The main Kings' Park units were built very close to the main building, which necessitated proper shoring of the excavation to prevent damage to the building due to settlement during construction. The groundwater table, too, was close to the surface, requiring continuous dewatering. However, since the units were located under the hotel lawn, they were not required to withstand heavy vehicle loads, which reduced the specific cost in comparison to the two former systems. All three systems, though discharging to the surface, were discharging in highly sensitive areas, environmentally, and the effluent needed to be of good aesthetic quality in order to be accepted by neighbouring residents. Hence, extra safety was included in the process designs, which is reflected in the costs.

At the other end of the spectrum, Ivy banks and Thilanka System 1a both show disproportionately low specific costs, with 3571 SLR/p.e. and 5161 SLR/p.e. respectively, including septic tanks. The low cost of Thilanka System 1.a. is due to the fact that it did not include an anaerobic filter and relied, instead, on a two stage VFPGF for secondary and tertiary treatment. The cost of anaerobic filter units is much greater in comparison to VFPGF's and percolation beds, as becomes evident later on in this section. Ivy banks, was the only system of the ones under consideration here, which was designed to discharge to ground. Therefore, it was designed for a much lower effluent quality, and incorporated a septic tank and anaerobic filter for the kitchen effluent only, which was a very small fraction of the total flow. Figure 4-27 and Figure 4-28 show a plot of the total system costs vs. p.e. for costs excluding septic tanks, and cost inclusive of septic tanks respectively. Both the figures have linear trend-lines shown on the plots. If the points corresponding to the special cases discussed above are ignored, the scatter is reduced to provide a reasonable fit.

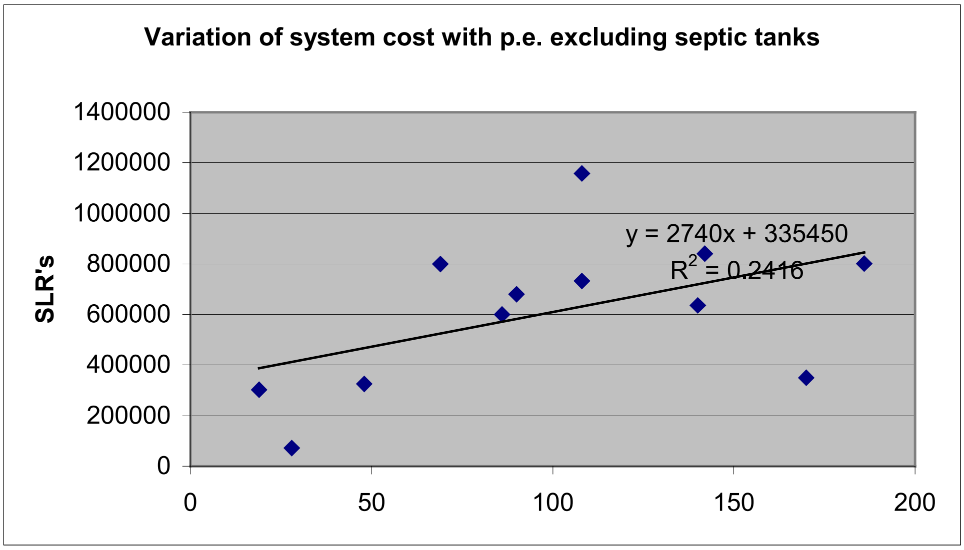

Figure 4-27. Plot of system cost vs. p.e. excluding septic tanks

In Figure 4-27 above, the two high cost points, one approaching SLR 1,200,000 and the other of SLR 800,000 correspond to the Devon Hotel and Kings Park systems. The two low cost points, one just below SLR 400,000 at 170 p.e, and the other, less than SLR 100,000 at 28 p.e. correspond to the Thilanka System 1.a. and the Ivy Banks system respectively. If these four points are ignored, the other points fit reasonably well around the trend-line, which implies a base cost of approximately SLR 335,000 for any system, plus an incremental cost of SLR 2740 per p.e. This model could be used to reasonably predict the cost of hotel systems incorporating anaerobic filters as a secondary treatment step, for population equivalents in excess of 50 p.e. Below this value the linear model loses its validity rapidly. Special cases with site constraints such as those described previously, would be exceptions, which would need to be evaluated independently. The same is true for Figure 4-28 below.

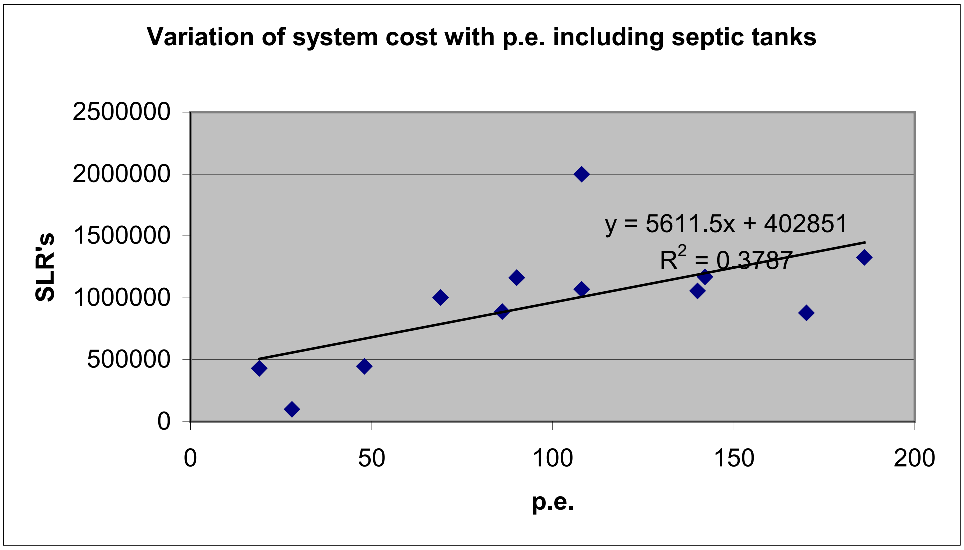

Figure 4-28. Plot of system cost vs. p.e. including septic tanks

Figure 4-29 shows a comparison of the average, minimum and maximum values of specific cost for each of the three cases, without considering the systems with special constraints, in the two latter cases.

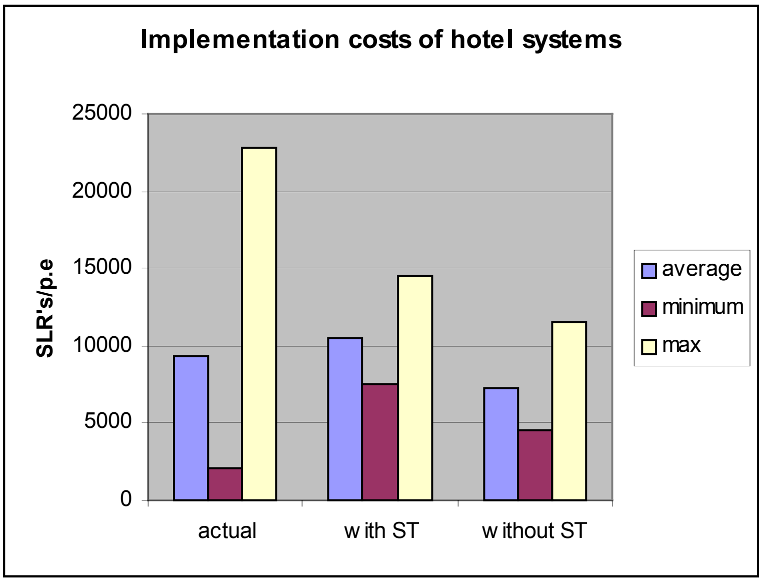

Figure 4-29. Comparison of specific implementation costs of hotel systems

The specific cost of systems, inclusive of septic tanks, varies around a mean value of 10481 SLR's per p.e., with a minimum of 7550 and a maximum of 14,552, while the specific cost without including septic tanks varies about a mean of 7224 per p.e., with a minimum of 4543 and a maximum of 11580 SLR's per p.e.

4.9.2. Houses

Table 4-4 below, shows the implementation costs for the individual house systems.

Table 4-4. Implementation costs for individual houses[27]

System	p.e.	Process	Effluent	With Septic Tank		Without Septic tank	
				Cost (SLR)	Cost/p.e. (SLR/p.e.)	Cost (SLR)	Cost/p.e. (SLR/p.e.)
Kadugannawa	5	st-rb-vf	Reuse	100000	20000	17000	3400
Nugegoda	10	st-af-pb	Discharge	200000	20000	89000	8900
Nawala	8	st-af-vf	Reuse	160000	20000	70000	8750
Kelaniya	10	st-af-pb	Discharge	130000	13000	85000	8500
Talwatte	5	st-af	Discharge	71000	14200	44000	8800
Moratuwa	10	st-af-pb	Discharge	115000	11500	70000	7000

All the systems happened to include septic tanks in their actual implemented form. The cost data excluding septic tanks has been compiled by simply deducting the actual implementation costs of the septic tanks in each case. As can be seen from the table, the specific cost figures show a greater degree of uniformity when considered without the septic tanks. This is because the septic tanks varied in material, with some being concrete tanks and the others being brick. The reason for the low figure of SLR 3400 per p.e. for the Kadugannawa system, is because it did not include an anaerobic filter for secondary treatment, for which it used a reed bed instead. The reed bed, though cheaper, has a larger footprint than an anaerobic filter, with a higher opportunity cost of land, which is not reflected in these figures. The specific costs are summarized in graphical form in Figure 4-30.

[27] Costs given are in Year 2000 SLR's with 76 SLR's per USD and 117 SLR's per GBP.

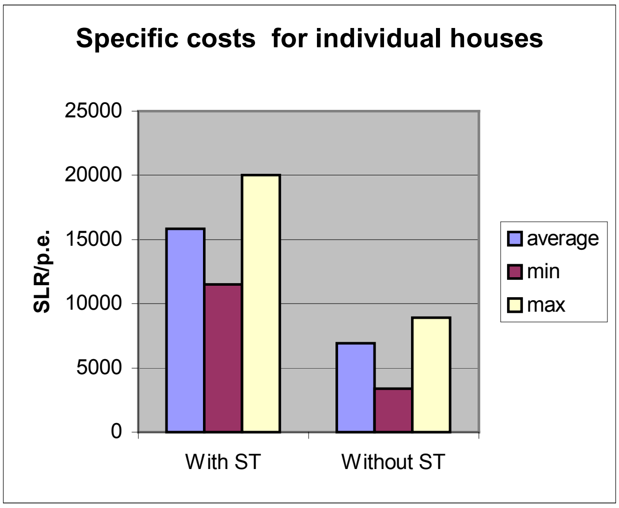

Figure 4-30. Specific costs for individual house systems

The specific cost including septic tanks vary from 11,500 SLR/p.e. to 20,000 SLR/p.e. about a mean value of 15,825, while the specific costs without septic tanks vary between 3400 and 8900 SLR/p.e. with a mean of 6933 SLR/p.e. These latter figures reflect the additional cost of implementing these systems had the houses already been equipped with septic tanks.

4.9.3. Housing Schemes

Table 4-5 below shows the implementation costs for housing schemes, with the figures adjusted to reflect the costs including and excluding septic tanks. The figures shown in italics represent hypothetical values.

Table 4-5. Implementation costs for housing schemes[28]

System	p.e.	Process	With ST		Without ST	
			Cost (SLR)	Cost/p.e. (SLR/p.e)	Cost (SLR)	Cost/p.e. (SLR/p.e)
Ranpok 1	115	st-af	*774000*	6730	407000	3539
Ranpok 2	135	st-rb	*955000*	7074	486000	3600
Luisa 1	344	st-rb	3250000	9448	1056000	3070
Luisa 2	172	st-rb	1700000	9884	575000	3343
Ceylinco 1	141	st-af-pb	1300000	9220	750000	5319
Ceylinco 2	79	st-af-pb	925000	11709	529000	6696
Poorwaram	275	st-af-vf	1650000	6000	1122000	4080

All the systems in this category discharged the effluent to streams except for the Ceylinco systems, which reused the effluent on-site for toilet flushing and gardening. The reason for the high specific cost of the Ceylinco systems was the fact that they were designed for effluent reuse, while the reason for the high specific cost, including septic tanks, for the Luisawatte systems was the fact that the septic tanks were part of a simplified and settled sewerage scheme and were designed to minimize the total project cost, inclusive of sewer network, rather than just minimize the cost of treatment.

[28] Costs given are in Year 2000 SLR's with 76 SLR's per USD and 117 SLR's per GBP.

Figure 4-31 shows a comparison of the average, minimum and maximum values of specific cost for the three cases.

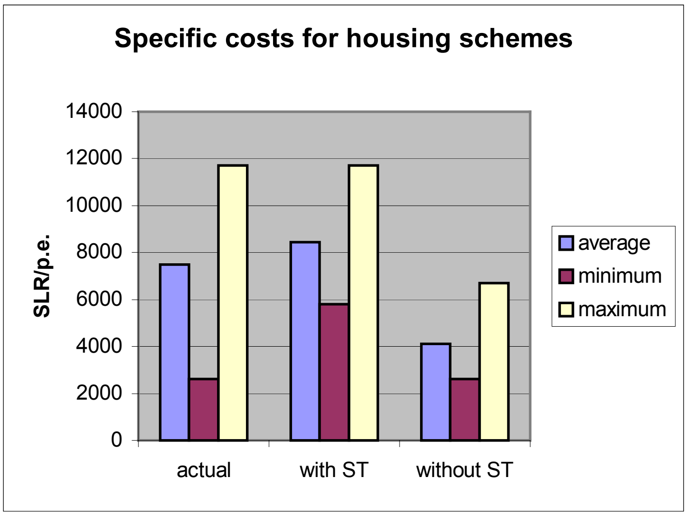

Figure 4-31. specific costs for housing schemes

As can be seen in the figure, the specific cost including septic tanks varied from 5800 SLR/p.e. to 11709 SLR/p.e. about a mean value of 8448 SLR/p.e., while the specific cost excluding septic tanks varied between 2609 SLR/p.e. and 6696 SLR/p.e. with a mean of 4102 SLR/p.e.

4.9.4. Schools and Halls of Residence

Table 4-6 below, shows the adjusted cost figures for schools and Halls of residence including and excluding septic tanks. The only effluent reuse system was the Kal Eliya system, which provided for reuse for gardening. All the other systems discharged to drains or streams

Table 4-6. Implementation costs for schools and housing schemes[29]

System	p.e.	Process	With ST		Without ST	
			Cost (SLR)	Cost/p.e. (SLR/p.e)	Cost (SLR)	Cost/p.e. (SLR/p.e)
Ladyhill	12	st-af	*205000*	17083	65000	5417
Akbar	160	st-rb	2700000	16875	620000	3875
Jayathileke	58	st-rb	1067000	18397	220000	3793
Kal Eliya	325	st-af-vf	1400000	4308	*986000*	3034

All the systems other than Ladyhill included septic tanks in the actual implementation, while Kal Eliya system included combined septic tank-anaerobic filter units, which were made in brick, and not designed to withstand vehicle loads. The septic tanks in all the other systems were made in reinforced concrete, which accounts for the high specific cost inclusive of septic tanks in comparison to the Kal Eliya scheme. This anomaly disappears when considering the specific costs without septic tanks. Figure 4-32 shows a comparison of specific costs for the three cases

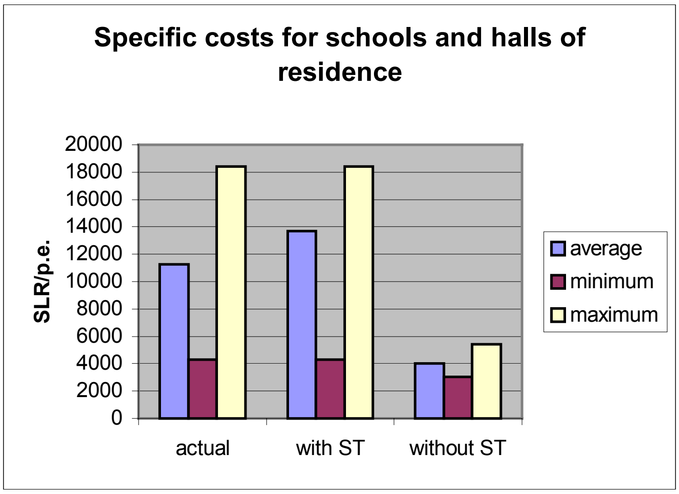

Figure 4-32. Specific costs for schools and halls of residence

As can be seen in the figure, the range of specific costs are the same for the actual implementation and the cost inclusive of septic tanks, with only the mean value increasing from 11,249 to 13,674 SLR/p.e. This is because only the value for

[29] Costs given are in Year 2000 SLR's with 76 SLR's per USD and 117 SLR's per GBP.

Ladyhill changes for the two cases. The specific cost without septic tanks varies between 3034 and 5417 SLR/p.e. with a mean of 4030 SLR/p.e.

4.9.5. Day-time occupancy buildings

Table 4-7 shows the costs for systems for daytime occupancy buildings including and excluding septic tanks. The numbers shown in italics are those values, which were calculated, based on hypothetical designs. The Avanhala, Seeduwa and Cold Stores systems actually had combined septic tank-anaerobic filter units. The costs excluding septic tanks for these systems are based on the costs of equivalent separate anaerobic filter units, without septic tanks.

Table 4-7. Implementation costs for day-time occupancy buildings[30]

System	p.e.	Process	With ST		Without ST	
			Cost (SLR)	Cost/p.e. (SLR/p.e)	Cost (SLR)	Cost/p.e. (SLR/p.e)
Avanhala	36	st-af	300000	8333	*124000*	3444
sampath	11	st-af	*135000*	12273	90000	8182
Seeduwa	50	st-af	625000	12500	*450000*	9000
Eng. Lib	18	st-af	*299000*	16611	165000	9167
PGIA	18	st-af	*143000*	7944	85000	4722
Cold stores	113	st-af-rb-vf	850000	7522	*567000*	5018

[30] Costs given are in Year 2000 SLR's with 76 SLR's per USD and 117 SLR's per GBP.

Figure 4-33 shows a comparison of the specific costs for the three cases.

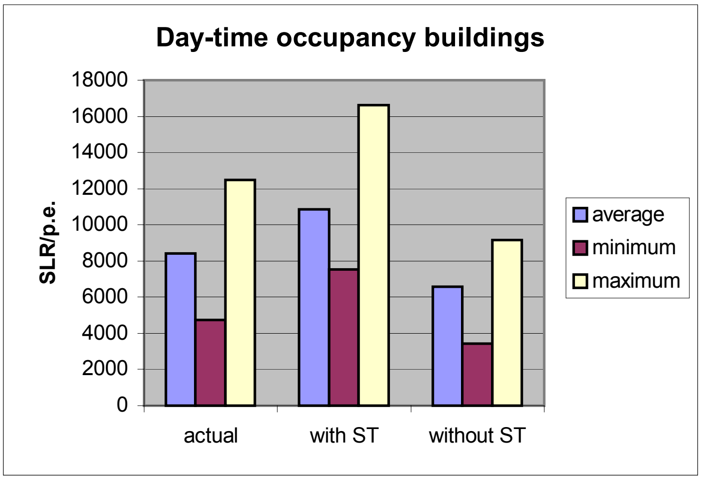

Figure 4-33. Specific costs for daytime occupancy buildings

The average actual specific cost of implementation of the systems was 8404 SLR/p.e. with a range of 4722 to 12,500 SLR/p.e. The specific cost inclusive of septic tanks varied from 7522 to 16,611 SLR/p.e. with a mean of 10,864 SLR/p.e., while the specific cost excluding septic tanks varied between 3444 and 9167 SLR/p.e. with a mean of 6589 SLR/p.e.

4.10. Unit process costs

4.10.1. Septic tanks

The main variables that affect the cost of septic tanks, other than population equivalent, are design sludge storage period and construction material. Brick tanks are significantly cheaper than tanks made out of reinforced concrete. Besides these main variables, the cost of septic tanks is affected to a lesser extent by whether they need to withstand vehicle loads or not and also on the configuration of the larger tanks. I.e. the number of parallel units in the tank, and their geometry. In order to compare the cost of septic tanks according to the change in the two principle variables, i.e. material and sludge storage period, a series of hypothetical tanks were designed and costed based on the experience gained through the implementation of the experimental full-scale systems. The tanks were designed assuming a total internal depth of 2.2 metres, which was typical in the real systems except when special site constraints such as shallow bedrock or very loose soil required shallower tanks. In the absence of such constraints, a depth of 2.2 metres was found to be an optimum compromise between cost and area occupied by the tank. The tanks were designed for optimum cost and efficiency, assuming that the space required for the particular geometry was available, without any special constraints. The results of this exercise are presented in Figure 4-34, which plots the cost of the tanks versus the p.e. for brick and concrete tanks separately for 1 year, 5 year and 10 year sludge storage periods.

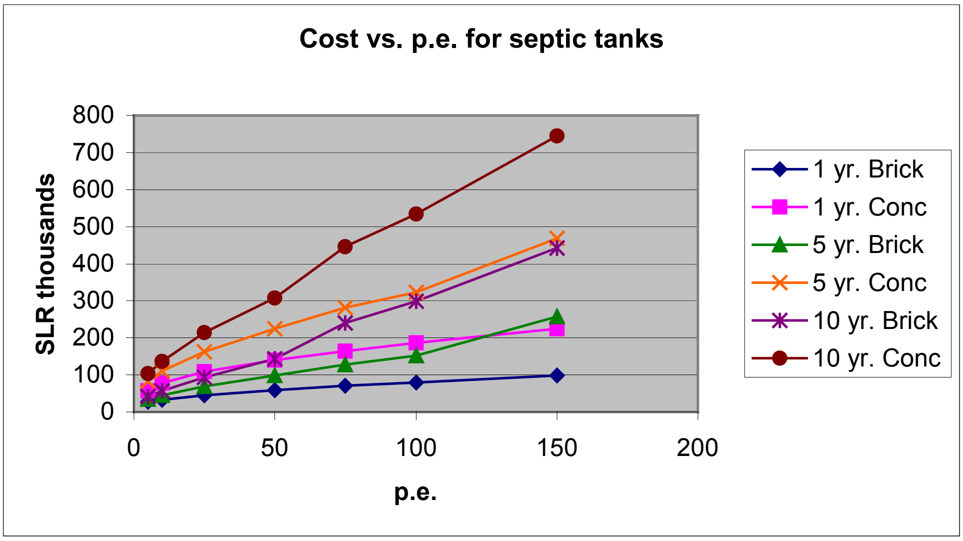

Figure 4-34. Variation of cost with p.e. for different types of septic tanks

The variation with p.e. shows a slight curve in cost at low population equivalents, which flattens out to a near linear variation above, typically, 50 p.e. The curves for the larger tanks also show an occasional discontinuity in the form of a jump in cost. These correspond to the points when the designs incorporated another parallel unit with increasing tank size.

Table 4-8 gives the costs for the different cases, which form the basis for the graphs in Figure 4-34.

Table 4-8. Cost of septic tanks in thousands of SLR's[31]

p.e.	1 year sludge storage		5 year sludge storage		10 year sludge storage	
	Brick	Concrete	Brick	Concrete	Brick	Concrete
5	27	57	35	83	42	103
10	33	78	45	111	57	136
25	45	109	69	163	93	214
50	58	140	99	224	143	307
75	70	164	127	281	240	446
100	80	186	152	323	299	534
150	99	224	258	469	443	744

The variation of specific cost with p.e. for the different cases is shown in Figure 4-35 for brick tanks and Figure 4-36 for concrete tanks.

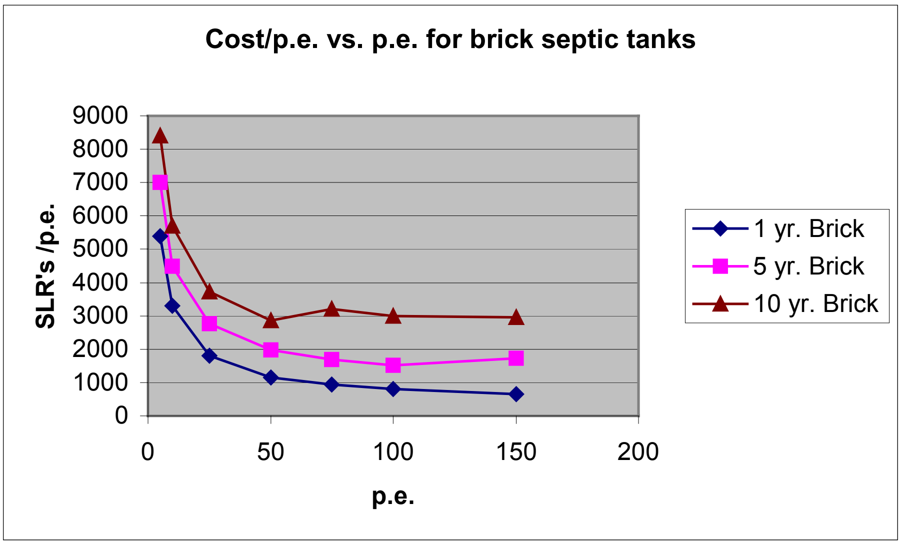

Figure 4-35. Variation of cost vs. p.e. for brick septic tanks

[31] Costs given are in Year 2000 SLR's with 76 SLR's per USD and 117 SLR's per GBP.

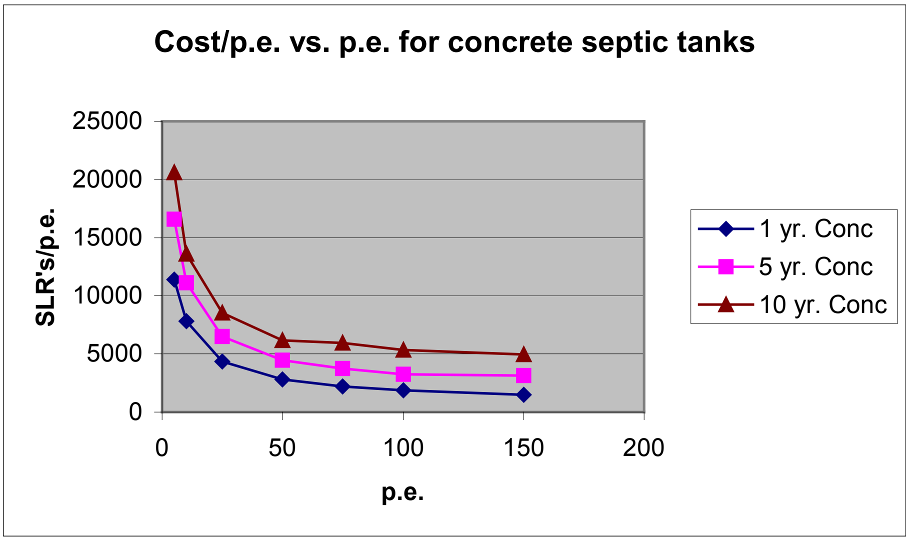

Figure 4-36. Variation of cost vs. p.e. for concrete septic tanks

As can be seen in the figures, the specific cost declines sharply with increasing p.e. at low population equivalents, tending to flatten out above, typically, 50 p.e.

4.10.2. Anaerobic filters

A similar exercise was carried out for anaerobic filters, as that for septic tanks in section 4.10.1 above. The main variable affecting the cost of anaerobic filters, other than construction material, is design hydraulic retention time. Anaerobic filters were designed and costed for similar cases to those of the septic tanks in the previous section, for three different nominal hydraulic retention times of 0.75 days, 1.0 day, and 1.5 days respectively. The filter depth was always 1.2 metres and comprised three equal layers of crushed rock of nominal size 50 mm, 25 mm and 12 mm, with the largest at the bottom and the smallest on top. An inlet chamber of 0.6 metre length was provided along the full width of the filter, and the overall internal depth of the unit was 1.9 metres. These conditions were typical in over 90 percent of the anaerobic filters designed for the actual full-scale systems. The filters were designed on the assumption that the optimum geometry could be implemented without special constraints on construction. Figure 4-37 shows the resulting variation of cost versus population equivalent for filters made in brick, while Figure 4-38 does the same for concrete filters

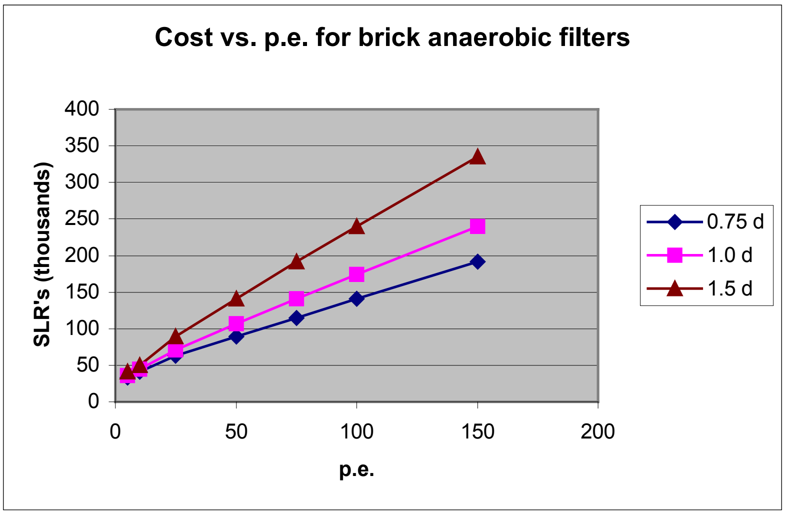

Figure 4-37. Variation of cost vs. p.e. for brick anaerobic filters

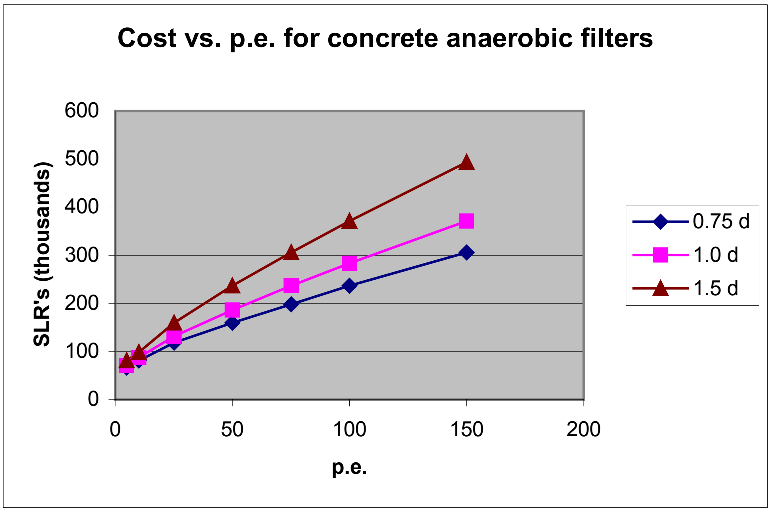

Figure 4-38. Variation of cost vs. p.e. for concrete anaerobic filters

Both the figures show a declining increment in cost with increasing p.e. for population equivalents less than 50, changing to a near linear variation above 50 p.e. Table 4-9 below, shows the cost figures in thousands of SLR's for the different hydraulic retention times for brick and concrete filters.

Table 4-9. Cost in thousands of SLR's for anaerobic filters of varying HRT[32]

p.e.	Brick			Concrete		
	0.75 d.	1.0 d.	1.5 d.	0.75 d.	1.0 d.	1.5 d.
5	33	36	41	67	71	81
10	41	45	50	81	88	99
25	63	71	89	119	132	159
50	89	107	141	159	186	237
75	115	141	192	198	237	306
100	141	174	240	237	283	371
150	192	240	335	306	371	493

Figure 4-39 below, shows the variation of specific cost with p.e. for brick filters and Figure 4-40 shows a similar graph for concrete filters.

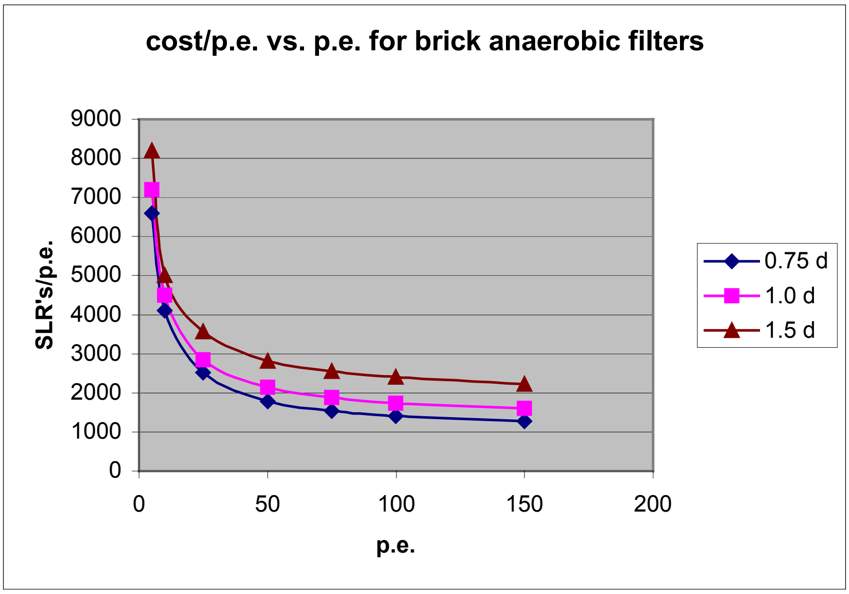

Figure 4-39. Variation of specific cost with p.e. for brick anaerobic filters

32 Costs given are in Year 2000 SLR's with 76 SLR's per USD and 117 SLR's per GBP.

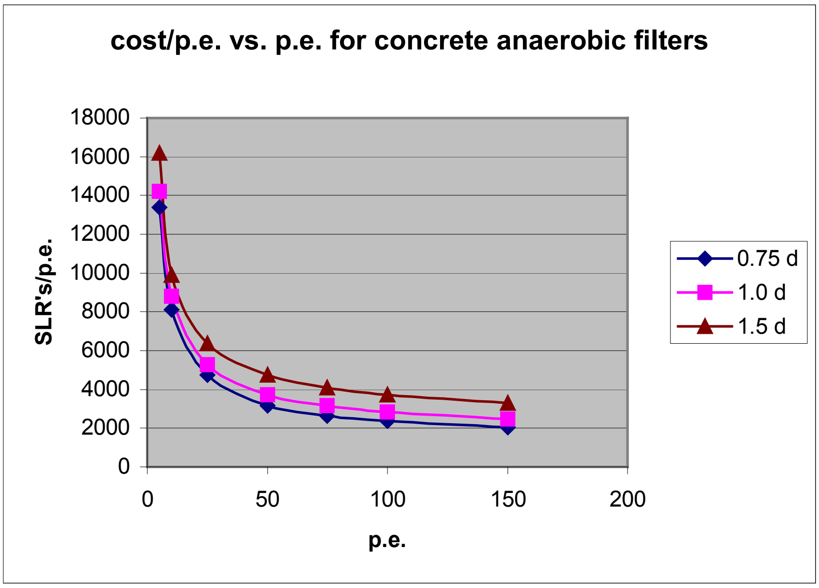

Figure 4-40. Variation of specific cost with p.e. for concrete anaerobic filters

As can be seen in the figures, the specific cost drops sharply with increasing p.e. at low population equivalents, with the rate of decline reducing as the population equivalent increases above 50.

4.10.3. Reed beds, percolation beds and VFPGF's

The cost of all these three types of unit processes varies in a similar manner, and is mainly dependent on bed area. For reed beds built entirely above ground, with the channels separated by masonry walls, as in the case of Akbar-Nell and Ranpokunugama systems, the cost is near constant at around 4000 SLR/m^2. For reed beds constructed below ground and contained within HDPE liners, the cost reduces to around 2500 SLR/m^2. In the case of VFPGF's and percolation beds, the cost is typically in the range of 3000 SLR/m^2, with an additional base cost of, typically, SLR 25000 for each pump and associated sump, switchgear etc. for pressurized systems. This additional base cost is usually present in VFPGF systems, which are often pressurized. The specific cost of these three unit processes, in terms of cost/p.e., varies with the particular specific area used for design. The design specific area ranged between 0.1 m^2/p.e. and 1.2 m^2/p.e. in the full-scale systems implemented.

4.11. Comparison of unit process costs and actual system costs.

Figure 4-41 and Figure 4-42 show a plot of the total system costs for the experimental full-scale systems vs. p.e. System costs have been estimated based on the unit process costs of their respective systems and compared against the actual system costs (Ref. Figure 4-27, page 191 and Figure 4-28, page 192). The unit process costs were estimated on the same bases and assumptions as described in Sections 4.10.1, 4.10.2, and 4.10.3 previously. I.e. based on the assumption that the most economical geometry and configuration could be implemented without site-specific constraints.

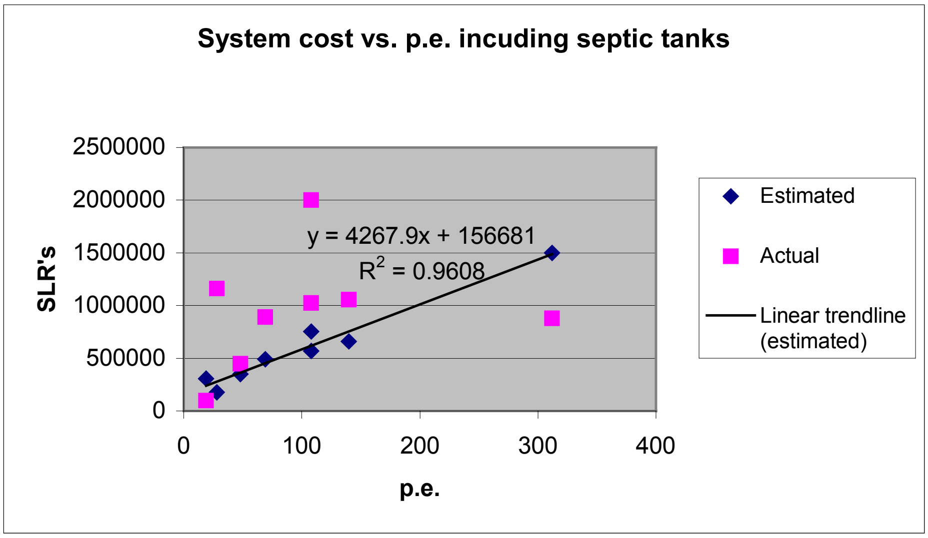

Figure 4-41. Plot of estimated and actual system costs vs. p.e. including septic tanks.

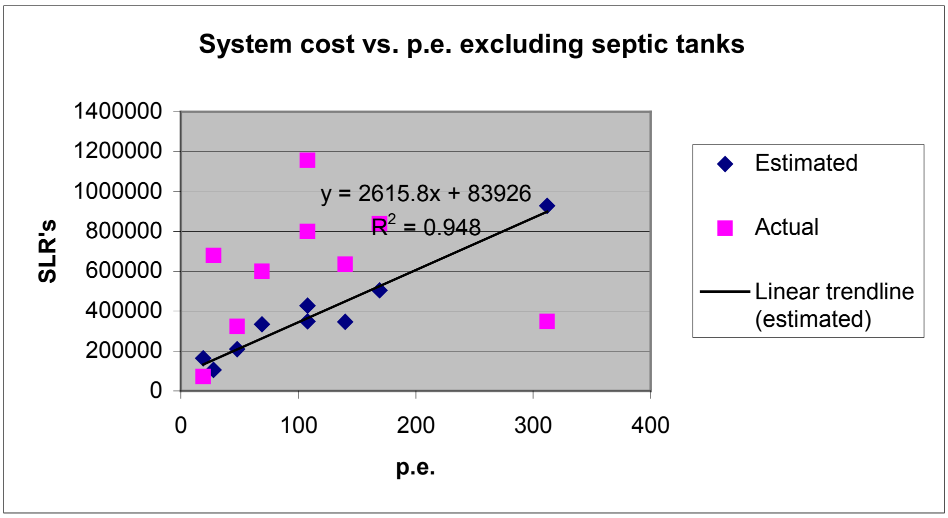

Figure 4-42. Plot of estimated and actual system cost vs. p.e. excluding septic tanks.

As can be seen in the figures, a linear trendline exists, of costs vs. p.e. with an intercept representing a base cost for each system. The variation of the actual costs from the trendline in the figure represents the effect of the various site-specific constraints particular to each individual case. Comparing the slopes of the trendlines in the figures with those of Figure 4-27 and Figure 4-28, i.e. the slopes of the trendlines of the actual costs of the systems, it is evident that the specific incremental cost for the estimated systems is approximately half that of the corresponding actual systems. However, no specific significance could be attached to this observation at present. What would be more useful, however, would be, given a specific site, to estimate the system cost based on the unit process costs after accounting for the site-specific constraints.

4.12. Evaluation of unit processes

4.12.1. Septic tanks

All the septic tanks built for the purposes of this study were designed according to the method described by Mara (1996). They all performed very well in terms of solids settling, with excellent removal of suspended solids, as was evident in the influent concentrations to the various secondary treatment units, which were invariably below 50 mg/l SS. The emptying cycle of the tanks designed according to this method was always slightly longer than the design emptying cycle. This was to be expected, as the systems were not always operating at full design capacity. The only variation of the practical performance of these tanks to the design hypotheses was that the accumulation rate of scum in tanks designed for hotel systems was greater than 40 percent of the sludge accumulation rate which is the usual value assumed in the design. This was increased to 50 percent in subsequent designs for hotels.

When comparing this method with that recommended by the Sri Lanka Code of Practice for septic tank design, the principal difference between the two methods is that the SLS code estimates the volume component required for sludge storage, and adds on the volume component for settling based on a hydraulic retention time of 1.5 to 2 days. Mara's method results in a much smaller additional volume component for settling, with an effective hydraulic retention time in the order of 0.5 days. However, an additional check for the minimum surface area required for settling is included. This is a more rational approach to the design in terms of settling, which is more dependent on surface overflow rate coupled with hydraulic retention time, rather than hydraulic retention time on its own. It also results in an overall lesser tank volume for population equivalents up to 150 p.e[33]. Figure 4-43 shows a comparison of tank volumes vs. p.e for 1-year tanks designed according to the UK and Sri Lanka codes of practice together with the method described by Mara.

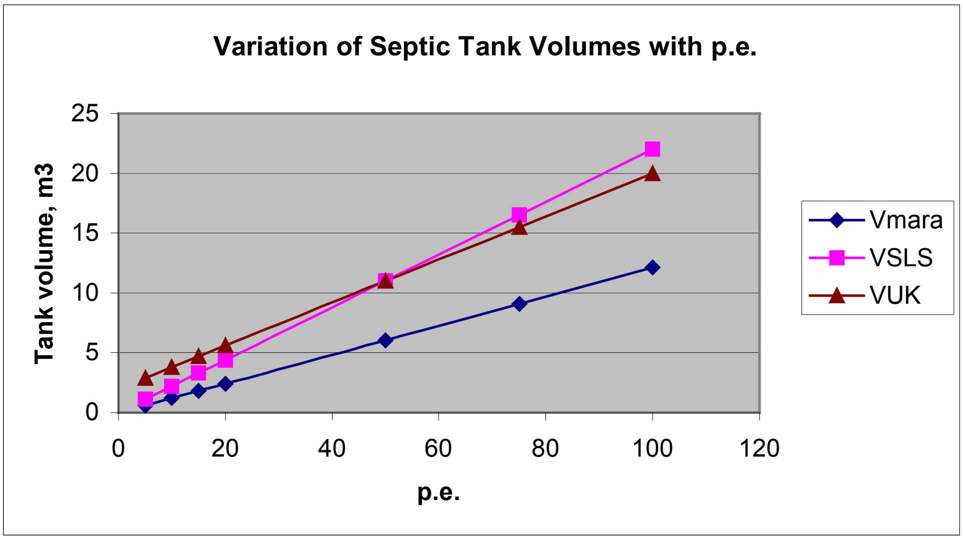

Figure 4-43. Variation of tank volume with p.e. for different design methods.

As can be seen in the figure, Mara's method requires significantly lower volumes than both the UK as well as Sri Lanka codes. The SLS code requires slightly lower volumes than the UK code up to 50 p.e, after which it requires larger volumes. Figure 4-44 shows a comparison of the required specific volumes (i.e. tank volume/p.e) for tanks designed according to the Sri Lankan code and Mara's method.

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[33] This is true for tanks with design emptying cycles of eight years or less.

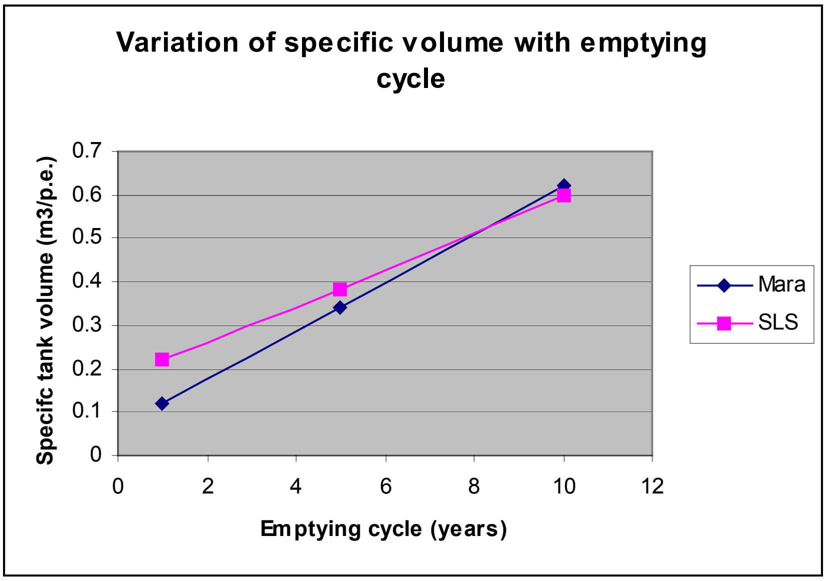

Figure 4-44. Comparison of specific volume variation with emptying cycle

As can be seen in the figure, Mara's method requires lower specific volumes for tanks with emptying cycles less than 8 years, with only a slight increase above the SLS volumes for tanks with 10-year cycles. For 1-year tanks, the SLS code requires almost double the volume with 0.22 m^3/p.e. as opposed to 0.12 m^3/p.e. required by Mara. For 5-year tanks the SLS method requires 12 percent more volume per p.e.

Figure 4-45 shows the percentage of tank volume allocated for sludge storage in the design for 1-year and 5-year tanks according to the two methods.

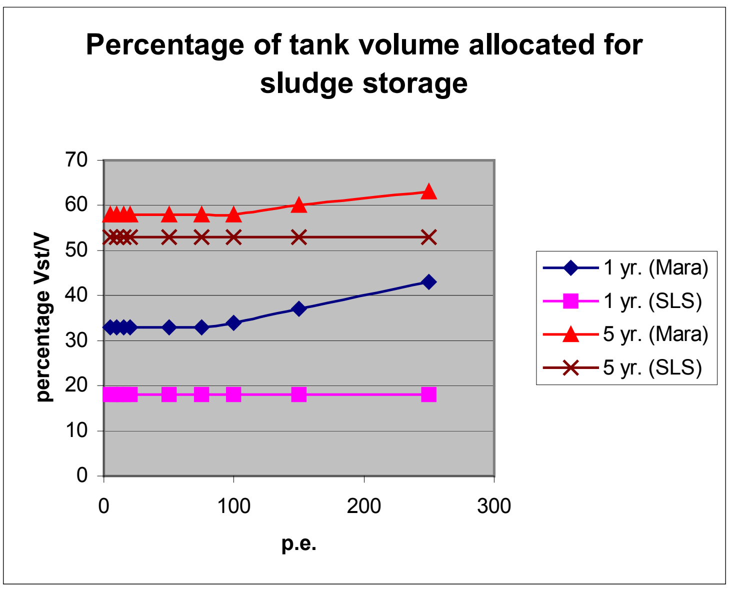

Figure 4-45. Percentage of tank volume allocated for sludge storage vs. p.e.

As can be seen in the figure, the SLS code allocates 18 percent of tank volume for sludge storage for 1-year tanks, and 53 percent for 5-year tanks. Mara's method allocates 33 percent for 1-year tanks and 57 percent for 5-year tanks up to 100 p.e. The values increase gradually beyond 100 p.e. in the case of the latter method. The Sri Lankan code recommends desludging the tank when it is one third full of sludge. This would mean that over 20 percent of the tank volume in 5-year tanks, which is allocated for sludge storage in the design, would remain permanently unutilised if the recommended maintenance practice were to be followed. This implies that 5-year tanks are over-designed by 20 percent in the SLS code according to its own basis. However, the practical reality is that tanks are often not desludged until almost completely filled with sludge.

The recommendations for desludging should logically be according to the design volume allocations. In the case of Mara's method, this would be when the tank is approximately one third full of sludge for 1-year tanks, half full of sludge for 5-year tanks and two thirds full of sludge for ten year tanks. Smaller tanks should be designed for longer emptying cycles, as they would be more likely to be emptied less frequently in practice. Also, even though this is a theoretical point with no real practical significance, it is interesting to note that tanks designed according to Mara's basis, would be operating within the design conditions for a significantly longer period of time, even in the case where tanks are not emptied according to the recommended frequency.

4.12.2. Anaerobic filters

All the anaerobic filters implemented in this study have proven to be robust and reliable in terms of both treatment and operation. It could be concluded therefore, that the anaerobic filter is an excellent secondary treatment unit process for septic tank effluents for the full range of applications under consideration in this study. They function well even under prolonged conditions of 'zero-maintenance' as was evident in the Ladyhill and Talwatte systems. Their only disadvantage is the higher capital cost of construction compared to the other secondary treatment unit processes, i.e. reed beds and VFPGF's. They should be always designed in upflow mode, as the down-flow variants could be prone to clogging due to gas entrapment in the filter bed as was experienced in the second stage of the Swiss Residence System 1 unit. Their performance is enhanced by being preceded by septic tanks because most of the first two steps of anaerobic decomposition, i.e. the acidogenesis and acetogenesis steps, which are mediated by relatively fast growing bacteria, would take place in the septic tank. This would allow more space and less competition for the much slower growing methanogens to establish themselves in the anaerobic filter in order to mediate the final stage of anaerobic degradation. This also effectively reduces the rate of biomass growth in the anaerobic filter bed. This is confirmed by the anaerobic filters monitored in this study, which did not show a continuous increase in the thickness of the attached and suspended biomass in the beds, beyond the first two years of operation, reaching what appeared to be a condition of final equilibrium. This would mean that anaerobic filters preceded by septic tanks, or any other anaerobic primary treatment process, would be less susceptible to clogging over the long term than those preceded by either aerobic processes, or physical treatment processes. This view is reinforced by the findings of Chernicharo and Machado, who reported good long-term performance of an anaerobic filter, which was preceded by an Upflow Anaerobic Sludge Blanket reactor (Chernicharo and Machado, 1998)

4.12.3. Reed beds

The reed bed units implemented under this study performed well as secondary and tertiary treatment units so far as treatment was concerned. However, they showed a fairly high degree of sensitivity to lack of maintenance. The maintenance requirements of reed beds, though minor, proved to be important to their long-term performance, as was seen in the case of the Akbar-Nell and Ranpokunugama systems. Consequently, they should be implemented only in situations where it could reasonably be expected that the vegetation would be maintained regularly, even at a very basic level. Secondary treatment reed beds were found to be susceptible to breeding of mosquitoes, particularly on the inlet end of the beds. In the case of the Kadugannawa reed bed, it was found that draining the bed approximately once a month reduced this problem considerably. Tertiary beds did not appear to suffer from this problem.

As far as design is concerned, the reed beds in this study were designed for a specific area of 0.4 - 0.6 m^2/p.e. However, the effluent from the 1st stage beds of the Akbar-Nell system, was found to be almost always within the surface discharge standards, even though the beds were severely overloaded, hydraulically, and not fully vegetated at the time. As was discussed in section 4.4.1,the beds were operating satisfactorily, treatment wise, at a hydraulic loading rate of 1.65 m/d and an organic loading rate of 81 $gBOD_5/m^2$.d. This translates to a specific area of 0.1 m^2/p.e on the basis of hydraulic loading, and 0.3 m^2/p.e on the basis organic loading for an influent concentration of 150 mg/l BOD_5. For an influent concentration of 50 mg/l BOD_5, the specific area on the basis of organic loading rate reduces to 0.1 m^2/p.e. Therefore, it could safely be concluded that a design specific area of 0.3 m^2/p.e could be used for secondary beds and 0.1 m^2/p.e for tertiary reed beds. This conclusion is reinforced somewhat by the excellent performance of the Kadugannawa secondary reed bed which was designed for 0.4 m^2/p.e.Though nitrification cannot be reliably predicted by the results of this study, ammonia nitrogen removals of over 80 percent at loading rates corresponding to 0.12 m^2/p.e have been reported for similar beds in Egypt (Williams et al, 1995). Williams et al also report 2 to 3 log reductions in faecal coliforms under similar conditions. Continued tests on the same beds have revealed their capability to remove shock loadings of helminth eggs of up to 500 eggs/litre, with most of the eggs being removed within the first few metres of the bed (Stott et al, 1999). Rash and Liehr (1999) have analysed the flow patterns in subsurface flow reed beds and found that vertical stratification of the flow in the beds causes a larger portion of the flow to take place along the bottom of the beds. This would imply that the use of smaller beds in series, as opposed to single large beds, would probably enhance the performance.

4.12.4. Percolation beds

All the percolation beds implemented in this study were gravity flow beds, used as tertiary treatment unit processes, mainly for polishing anaerobic filter effluent for reuse or discharge into sensitive water bodies, except for the Kadugannawa unit which was receiving effluent from a secondary reed bed. The design specific area ranged from 0.11 to 0.5 m^2/p.e with most of the beds designed for 0.2 m^2/p.e. The typical bed depths were either 0.9 metres or 0.6 metres, with a couple going up to 1.5 metres. Their performance in the main has been within expectations, and very satisfactory. The filter media used has either been sand / gravel retained on a 2 mm sieve, or stone chips. All the beds other than the Kadugannawa bed have been closed beds, buried under driveways, garages, and the like. Though most of the beds had draft tubes for passive ventilation of the bed, their contribution is dubious. The percolation bed of the Swiss Residence System 1 functioned very satisfactorily without draft tubes for two years. Subsequent installation of draft tubes did not result in a significant change in performance.

Observation of the Kadugannawa bed in operation showed that the flow in gravity flow beds is not distributed effectively to utilize the full area of the bed, with a large portion of the bed being unutilised due to lack of proper flow distribution. Therefore, effective distribution of flow would be the main requirement to increase the efficiency and reduce the size of these beds further. Pressurized systems, however, with a properly designed distributor would have a much more efficient distribution of flow, and therefore could be designed with lower specific areas. This is in agreement with the findings of Fazio et al (1993) who observed significant improvement of performance of secondary percolation beds treating septic tank effluents with careful control of flow distribution.

Although, too many conclusions cannot be drawn regarding the treatment efficiency of percolation beds, as a unit process, from this study, they have by and large proven to effectively serve the purpose for which they have been applied. Their main role, in the case of effluent discharge applications would be to reduce any remaining odour from the anaerobic filter effluent, while buffering out any violations of discharge limits for BOD, ammonia nitrogen and suspended solids. In the case of effluent reuse applications, they serve an important role in removal of faecal coliforms. Assuming no significant removal of coliforms in the anaerobic filter unit, and an influent concentration of the order of 10^5 cfu/100ml, the effluent quality provided by the Swiss Residence percolation bed would indicate a 3-log removal of faecal coliforms by the bed. This is in general agreement with most of the French authors on percolation beds who invariably report a faecal coliform removal of over 3 logs (Fazio et al, 1993, Guilloteau et al, 1993, Brissaud et al, 1999). They also found excellent removal of Helminth eggs (Guessab et al, 1993). Brissaud et al, in particular, report practical removal of faecal coliforms in excess of 3 log cycles, for hydraulic loadings corresponding to less than 0.2 m^2/p.e. Guilloteau et al (1993) found that the most effective part of the bed is in the top 30 centimetres; implying bed depths in excess of 0.5 metres are not necessary. This was, however, for beds using sand. For beds packed with stone chips, which is a more freely available material in Sri Lanka, a slightly larger depth of abut 0.9 metres would probably be safer, particularly for pathogen removal.

4.12.5. VFPGF's

All the VFPGF units implemented in this study, bar two, have been as tertiary treatment unit processes. The design specific area has ranged from 0.1 m^2/p.e to 0.7 m^2/p.e, with a value of 0.2 m^2/p.e being typical. A value of 0.1 m^2/p.e was used for VFPGF units at King's Park Hotel and Ceylon Cold Stores. In both these systems, the VFPGF unit was actually a quaternary treatment unit process, with a 0.1 m^2/p.e percolation bed preceding it in the case of King's Park, and a 0.1 m^2/p.e reed bed preceding it at Cold Stores. The King's Park unit has been functioning well, even at this level of hydraulic loading. The Cold stores unit has not been commissioned as yet. The two VFPGF's used as secondary treatment unit processes were the units in Swiss Residence, System 2, and Thilanka, System 2. The Swiss Residence unit was designed for a specific area of 0.4 m^2/p.e, while the Thilanka unit was designed for 0.2 m^2/p.e. Both the units suffered problems with clogging.

Many authors attribute clogging of vertical flow beds to high organic loading rates (Laak, 1986, Loffler, 1992, Platzer and Mauch, 1997). Loffler (1992) recommends a maximum organic loading rate of 40 g. BOD/ m^2.d to avoid soil clogging for vertical flow beds in Europe. Platzer and Mauch (1997) recommend a maximum value of 25 g COD/m^2.d, which works out to 42 gBOD/m^2.d if one assumes a BOD / COD ratio of 0.6 which is typical for domestic effluents. They further postulate that the influent concentration is more a factor in clogging than organic loading rate per se. However, they do not specify a maximum concentration, but a maximum organic loading rate.

The organic loading rate of the Swiss Residence VFPGF was 12.3 $gBOD/m^2.d$ during the start up period. No problems with clogging occurred then. During phase 1 operation, the organic loading rate rose to an average of 109 $gBOD/m^2.d$, with an average influent concentration of 109mg/l BOD_5. During phase 2, the average organic loading rate was 69.3 $gBOD/m^2.d$, which then doubled to 140 $gBOD/m^2.d$ after the connection of the laundry wastewater to the system. The problem of clogging manifested itself during phase 2, when the average organic loading rate was 69 $gBOD/m^2.d$ and the hydraulic loading rate was less than the design value. In the Thilanka units, no clogging problems arose when the units were operated as tertiary units, and clogging problems manifested after septic tank effluent was connected directly to the units in the final stages of commissioning. The clogging first appeared, and was significantly more serious, in the second stage beds, which were much smaller than the first stage beds. This does not support Platzer and Mauch's claim that influent concentration is more a factor than organic loading rate in clogging, since the first stage beds would have had a higher influent concentration than the second stage beds. The only difference between the two stages was that the second stage units were significantly smaller than the first stage, and consequently, could have had a higher organic loading rate. However, the Swiss Residence VFPGF did not clog at the higher organic loading rate of 109 $gBOD/m^2.d$ when the influent concentration was lower at 109 mg/l BOD_5, but it clogged at a lower organic loading rate of 69.3 $gBOD/m^2.d$ when the average influent concentration was higher at 324 mg/lBOD_5. This would suggest that probably both, organic loading rate and influent concentrations are factors in clogging.

Both the above VFPGF units were experiencing other stresses at the time clogging manifested. The Swiss Residence unit was experiencing very low influent pH, while the Thilanka units were almost devoid of vegetation due to marauding monkeys. However, based on the above experience, it would be safe to recommend that the organic loading rate for VFPGF's in Sri Lanka be limited to a maximum of 60 $gBOD/m^2.d$, until further information is available. This would work out to minimum specific areas of 0.13 m^2/p.e for tertiary units and 0.4 m^2/p.e for secondary units, based on average influent BOD_5 concentrations of 50 mg/l and 150 mg/l respectively. For hotel systems handling only kitchen wastewater, the minimum specific area would be 0.7 m^2/p.e, based on an average influent concentration of 250 mg/l. This value is supported by the performance of the Coral Sands unit, which has a specific area of 0.7 m^2/p.e and is operating satisfactorily. However, in such systems, the bed should incorporate a layer of limestone as well, in order to raise the pH, as was the case at Coral Sands. This is important for nitrification too, as it does not occur at pH less than 5.6 in vertical flow beds (Sun et al, 1999). However, systems treating kitchen wastewater in combination with other black and grey water can be designed as regular units, as was the case at Thilanka. Smaller values of specific area, down to 0.1 m^2/p.e could be used for quaternary VFPGF's to achieve effective nitrification where necessary, as was evident at King's Park. No problems occurred with mosquitoes in any of the units, unlike the horizontal flow reed beds. This was probably due to the lack of standing water in the vertical flow units.