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Performance of an anaerobic baffled reactor and hybrid constructed wetland treating high-strength wastewater in Nepal—A model for DEWATS

Shirish Singh[a,*], *Raimund Haberl*[a], *Otto Moog*[b], *Roshan Raj Shrestha*[c], *Prajwal Shrestha*[d], *Rajendra Shrestha*[d]

[a] *Institute for Sanitary Engineering and Water Pollution Control, University of Natural Resources and Applied Life Sciences (BOKU), Muthgasse 18, A-1190 Vienna, Austria*

[b] *Institute of Hydrobiology and Aquatic Ecosystem Management, University of Natural Resources and Applied Life Sciences (BOKU), Austria*

[c] *UN-HABITAT, Water for Asian Cities Programme, Nepal*

[d] *Environment and Public Health Organization (ENPHO), Nepal*

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ABSTRACT

Centralized wastewater treatment systems require sophisticated technologies and skilled manpower for their operation and maintenance (O&M). These systems have huge construction as well as O&M costs. Therefore, a Decentralized Wastewater Treatment System (DEWATS) rather than a centralized system might be especially beneficial in developing countries. A model for DEWATS is developed in Nepal with Anaerobic Baffled Reactor (ABR) and hybrid Constructed Wetland (CW). The DEWATS treats high-strength wastewater from 80 households (400 PE). This paper summarizes the performance of the DEWATS from July 2006 to August 2007 in the removal efficiencies of TSS, BOD_5, COD, NH_4–N, TP and FC. The ABR is very effective in the removal of organic pollutants and could achieve TSS removal up to 91%, BOD_5 up to 78% and COD up to 77%. The average removal efficiencies of the DEWATS is 96% TSS, 90% BOD_5, 90% COD, 70% NH_4–N, 26% TP and 98% FC.

1. Introduction

Centralized wastewater collection and treatment systems are not the most cost-effective or environmentally sound option for all situations (e.g., sewage treatment plants can discharge high point source loadings of pollutants into receiving waters). Centralized wastewater treatment plants require conventional (intensive) systems, which rely on sophisticated technologies and plants operation by highly skilled personnel. They are often infeasible or cost prohibitive, especially in areas with low population and dispersed households (U.S. EPA, 2002). The construction and operation and maintenance (O&M) of conventional wastewater treatment plants require large amounts of money that countries facing structural and financial adjustment cannot afford (Kengne Noumsi et al., 2005a,b).

The Decentralized Wastewater Treatment System (DEWATS) rather than a centralized system might be especially beneficial in developing countries and allow locals to

* *Corresponding author*. Tel.: +43 1 36006 5834; fax: +43 1 3689949.
E-mail address: h0540855@edv1.boku.ac.at (S. Singh).

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deal with their situation when there is a lack of action or capacity by the central governing body (Green and Ho, 2005). The DEWATS is widely used not only in developing countries, but in developed countries as well. In Germany, approximately 10% of the population discharges their wastewater to a DEWATS and the percentage increases to 30% in certain federal states, particularly in Eastern Germany (Kegebein et al., 2007). In the United States, more than 60 million people depend on decentralized systems (U.S. EPA, 2002).

The DEWATS is better suited to translate Bellagio Principles No. 3 (perceiving human excreta and wastes as potential resources) and No. 4 (solving sanitation issues as close as possible to the source of waste generation) into practice (EAWAG, 2008). The DEWATS is mostly implemented with natural (extensive) systems in the developing countries although a variety of intensive systems like membrane filtration, sequencing batch reactor, etc. are also used in the developed countries. The major advantages of the DEWATS with extensive systems are as follows (Sasse, 1998; Brissaud, 2007):

- Reliable, robust and buffer shock loads;
- No (or very little) energy is required;
- Limited sludge production;
- O&M does not require highly skilled personnel;
- Very low O&M cost;
- Reduces the risks associated with system failure;
- Increases wastewater reuse opportunities;

The DEWATS is not the best solution everywhere. However, where skilled and responsible O&M cannot be guaranteed, the DEWATS is undoubtedly the best choice available (Sasse, 1998).

Wastewater pretreatment in high-rate anaerobic reactors like UASB and post-treatment by CW have already been investigated and promising results have been reported (Alvarez et al., 2008; Barros et al., 2008). The use of ABR as pretreatment has not yet been investigated. The applicability of ABR as high-rate anaerobic reactor for pretreatment and secondary treatment through CW for DEWATS has been investigated in this study. The main advantages of this combination were assumed to be:

- Reducing the complexity in the construction, O&M of high rate anaerobic reactors;
- Reducing SS removal to reduce the clogging in the following CW; and
- Reducing the sizing of the CW.

2. Materials and methods

As a part of the Pilot and Demonstration Activities (PDA) program for water, the Asian Development Bank (ADB)-managed program, the first community-scale DEWATS was constructed in Nepal. UN-Habitat and WaterAid, Nepal, provided co-funding for performance monitoring of the DEWATS and dissemination. The DEWATS was constructed under the technical supervision of the Environmental and Public Health Organization (ENPHO). The DEWATS treats an average daily flow of 10 m^3/d wastewater from about 80 households (400 PE) at a peri-urban area in Thimi Municipality. Components and layout plan of the DEWATS is shown in Fig. 1.

2.1. *Preliminary treatment*

The preliminary treatment consists of a coarse screen and a grit chamber.

2.2. *Primary treatment*

The primary treatment of wastewater is achieved in an ABR because of its huge potential in removing the pollutants (Barber and Stuckey, 1999; Wanasen, 2003; Foxon et al., 2004). The design criteria for the ABR were taken as 4 h retention time for sedimentation, 3 months sludge digestion period and 1 year for sludge withdrawal. The dimension of the ABR was calculated as 6 m × 4 m × 2 m (L × B × H) resulting in 1.2 days HRT when the reactor is two-thirds filled with sludge. The walls of the ABR and the hanging baffles are constructed of reinforced cement concrete whereas the other baffle walls are constructed of brick masonry. The inlet is placed 1.75 m from the bottom of the ABR and the outlet slightly below the inlet, resulting in an effective volume of 42 m^3. Holes at the bottom of each compartment of the ABR are made for easy sludge withdrawal and closed with a PVC pipe.

2.3. *Secondary treatment*

The secondary treatment of wastewater is carried out in hybrid CWs. In the first stage, the wastewater is treated in two Horizontal Flow Constructed Wetlands (HFCWs) and then in two Vertical Flow Constructed Wetlands (VFCWs) (Fig. 1). Collecting chambers are placed after the ABR and each CW to control the flow to the required cell of the wetlands.

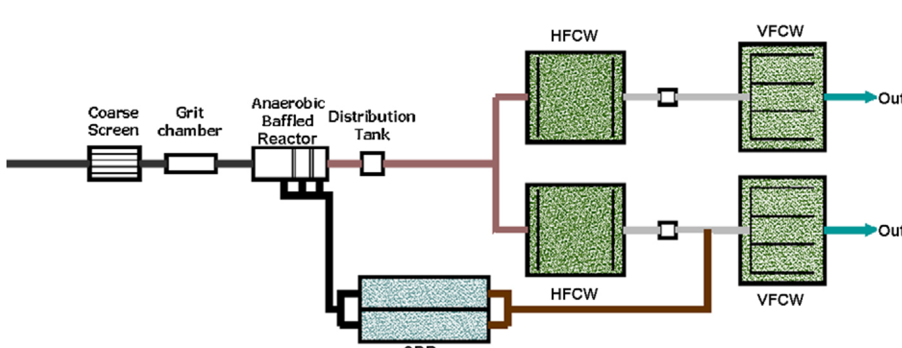

Fig. 1 – Plan of the wastewater treatment plant.

2.3.1. HFCW

The HFCW was designed based on the first-order decay rate (Eq. (1)).

$$A_h = \frac{Q_d(\ln C_i - \ln C_e)}{K_{BOD}} \quad (1)$$

A_h = Surface area of bed (m^2), Q_d = average daily flow rate of sewage (m^3/d), C_i = influent BOD_5 concentration (mg/l), C_e = effluent BOD_5 concentration (mg/l), K_{BOD} = rate constant (m/d).

The reaction rate constant of 0.13 m/d was used and it was expected to remove about 70% of the BOD_5. Two HFCWs with a total area of 150 m^2 (about 8 m × 9.5 m each) and a depth of 0.4–0.5 m received wastewater from the ABR. The depth at the inlet of the HFCW is 0.4 m and with a bed slope of about 1%, the depth of HFCW at the outlet is 0.5 m. An average depth of 0.45 m was used taking into consideration the precipitation, which could cause surface flow, and that shallow HFCW with an average water depth of 0.27 m was more effective than deep HFCW with an average water depth of 0.5 m (Garcia et al., 2005). The walls of the HFCW are constructed of brick masonry and the wetland is sealed with a plastic liner of 500 μm thickness. 20–40 mm washed gravel is used as substrate in the inlet and outlet zones, whereas, 5–10 mm washed gravel is used in the treatment zone. The wastewater is distributed through a 150-mm diameter pipe with 20 mm diameter perforations at a distance of 150 mm centre to centre in one HFCW. In the other HFCW, slots (300 mm × 30 mm) in 150 mm pipe at a distance of 75 mm in between distribute the wastewater. The inlet pipe is placed just above the substrate. The drainage pipe is 150 mm diameter pipe with 6 mm perforations. One HFCW is planted with *Phragmites karka* and the other with *Canna latifolia*. The HFCWs are operated with continuous loading.

2.3.2. VFCW

VFCW was also designed as per Eq. (1) but a higher rate constant value of 0.15 m/d was used. Two VFCWs with a total area of 150 m^2 (10 m × 7.5 m each) with a depth of 0.55 m received wastewater from the HFCWs. The walls of the VFCW are constructed of brick masonry and the wetland is sealed with a plastic liner of 500 μm thickness. The substrate arrangement of the VFCW is as follows:

Layer	Thickness	Size of substrate
From top to bottom		
Protection layer	5 cm	5–10 mm gravel
Main layer	30 cm	0–4 mm coarse sand (d_{10} = 0.35 mm and d_{60}/d_{10} = 3.3)
Transition layer	5 cm	5–10 mm gravel
Drainage layer	15 cm	10–20 mm gravel
Total	55 cm	

Most VFCWs in the UK are built 0.5–0.8 m deep (Cooper et al., 1996). In contrast to that, depth greater than 0.5 m for the main layer is recommended in Germany and Austria (ÖNORM, 2005; DWA, 2006). Still greater depth of 1 m is recommended in Denmark (Brix and Arias, 2005). Despite the fact, a shallower depth of 0.55 m was used in order to compromise the cost of the VFCW and because the depth recommended might not be required in the subtropical climate.

The wastewater is distributed through a network of four 100 mm diameter pipe connected to a feeding tank (1.5 m^3 per feed). 6 mm holes are made in the pipes at a distance of 1 m centre to centre. The treated wastewater is collected through a network of drainage pipes, which consists of four 100 mm diameter perforated pipes with 6 mm diameter perforations. The VFCWs are planted with *Phragmites karka* and are operated with intermittent loading, which is maintained hydro-mechanically. The duration of intermittent loading was about 5 min. It was observed that the minimum time interval of was about 30 min and a maximum of about 2 h during loadings.

2.4. Sludge treatment

CW Sludge Drying Bed (SDB) was designed to treat the primary sludge with a maximum sludge application depth of 30 cm. Two CW SDBs with a total area of about 70 m^2 (12 m × 3 m each) and depth of 50 cm received stabilized sludge from the ABR, which resulted in sludge application load of 30 kg TS/m^2/year. The percolate from the SDB is further treated in one of the VFCWs (Fig. 1). The walls of the SDB are constructed of brick masonry (one side is constructed of reinforced cement concrete) and are sealed with a plastic liner of 500 μm thickness. The substrate arrangement of the SDB is as follows:

Layer	Thickness	Size of substrate
From top to bottom		
Main layer	30 cm	0–4 mm coarse sand
Transition layer	05 cm	5–10 mm gravel
Drainage layer	15 cm	20–40 mm gravel
Total	50 cm	

2.5. Analysis

Grab samples of wastewater were collected from every treatment stage (Fig. 1) and were analyzed for TSS, BOD_5, COD, NH_4–N, TP and FC at ENPHO laboratory. Sometimes, only one HFCW or only one VFCW was operated for maintenance of the CWs. The concentrations of the pollutants in the CWs represent average values in the wetlands.

The ABR was desludged on 13 June 2007. Sludge is first treated in SDBs and percolate from the SDBs is treated in one of the VFCWs (Fig. 1). The sample of sludge (raw) was collected almost in the middle of the desludging time to have a representative sample of raw sludge. The sample of the percolate from SDBs was collected after about an hour of complete desludging of the ABR. Likewise, the sample of treated percolate from the VFCW was collected after 1 h of loading into the VFCW. The samples were analyzed for TSS, BOD_5, COD and TKN. All parameters were analyzed in accordance with the procedures described in Standard Methods (APHA, AWWA, WEF, 1998).

Tracer study was carried out on 22 and 23 December, 2006 to ascertain the daily flow and the nominal Hydraulic Retention Time (HRT) in the wetland. 5 kg of NaCl was dissolved in 25 l of tap water and fed into the VFCW through the inlet distribution network at 13:00 on 22 December. The effluent conductivity was measured every 15 min until 23:00 and started again at 05:00 on 23 December till 08:00. 5 kg of NaCl was dissolved in 30 l of tap water and fed into the HFCW through the inlet pipe at 08:45 on 23 December. The effluent conductivity was measured every 15 min till 19:30.

2.6. Construction cost

The total cost of the DEWATS was about US$ 31,500 whereas the costs of wetlands only amounted to about US$ 18,000. The total specific cost of the DEWATS was calculated to be about US$ 80/PE, whereas the specific cost of the wetland only was calculated to be about US$ 60/m^2.

2.7. O&M cost

A caretaker is assigned for the O&M of the DEWATS. Regular maintenance works consisted of daily inspection of coarse screen and grit chamber and cleaning if required; weekly removal of unwanted vegetation from the wetlands and monthly cleaning of the wetland inlet/outlet systems. The harvesting of the vegetation is carried out twice a year. The primary treatment is desludged once a year. The O&M cost is about US$ 520 per annum.

3. Results and discussions

3.1. Tracer study

Fig. 2 shows the cumulative effluent conductivity plotted against time. The theoretical HRT for HFCW is calculated to be 24.2 h but the tracer study showed a nominal HRT of only 5.2 h. Similarly, the theoretical HRT for VFCW is calculated to be 29.7 h but the tracer study showed a nominal HRT of only 5.6 h.

The hydraulic efficiency is a measure to assess the hydrodynamic performance of detention systems against the uniformity of flow and the effective utilization of the available detention storage volume. It represents the well distribution of wastewater through the wetland. Occurrence of short-circuits and poor utilization of available detention storage results in poor hydraulic efficiency. Use of proper shape and depth of the wetland, locations and types of inflow and outflow structures are major factors for a good hydraulic efficiency. The Hydraulic Efficiency (λ) of the wetland was calculated using the equation (Persson et al., 1999):

$$\lambda = e\left(1-\frac{1}{N}\right) = \left(\frac{t_{mean}}{t_n}\right)\left(1-\frac{t_{mean}-t_p}{t_{mean}}\right) = \frac{t_p}{t_n}$$

where, λ = hydraulic efficiency; $e = (t_{mean}/t_n)$; $N = (t_{mean}/t_{mean} - t_p)$; $t_{mean} = t_{50}$ = 50th percentile of the hydraulic residence time distribution; t_p = time to peak; and t_n = nominal HRT.

Based on the hydraulic efficiency, the wetland is categorized as having: (i) good hydraulic efficiency with $\lambda > 0.75$; (ii) satisfactory hydraulic efficiency with $0.5 < \lambda \leq 0.75$; and (iii) poor hydraulic efficiency where $\lambda \leq 0.5$ (Persson et al., 1999). The hydraulic efficiency of HFCW was 0.66 and was categorized with satisfactory hydraulic efficiency. Improper distribution of wastewater in the influent and clogging in some parts of the inlet zone could be the reason for lower hydraulic efficiency in the HFCW. The hydraulic efficiency of VFCW was 0.80 and was categorized with good hydraulic efficiency. In contrast to the HFCW, proper distribution of wastewater through out the wetland area and lower TSS loading could be the causes for higher hydraulic efficiency in the VFCW.

3.2. Performance

Table 1 shows the parameter concentrations in each stage of the treatment. Six samples were analyzed during a period of July 2006 to August 2007. The influent concentrations of parameters were found to be high. The influent concentrations of TSS, BOD_5, COD, NH_4–N and TP were found to be 1506 ± 1607, 1593 ± 686, 2914 ± 1405, 142 ± 20 and 24.4 ± 7.6 mg/l respectively. The wastewater could be categorized as high strength wastewater (Metcalf and Eddy, 2003). The high concentration of parameters is due to less water usage in the catchment of the DEWATS. It was calculated that the specific wastewater produced is only about 25–30 l/capita d. Considering an average 40 g BOD_5/capita d is produced by an individual, the BOD_5 concentration ranges from 1333 to 1600 mg/l.

Table 2 shows the percentage removal efficiencies of parameters in each stage of the treatment. The removal efficiencies of the organic parameters in the ABR are high

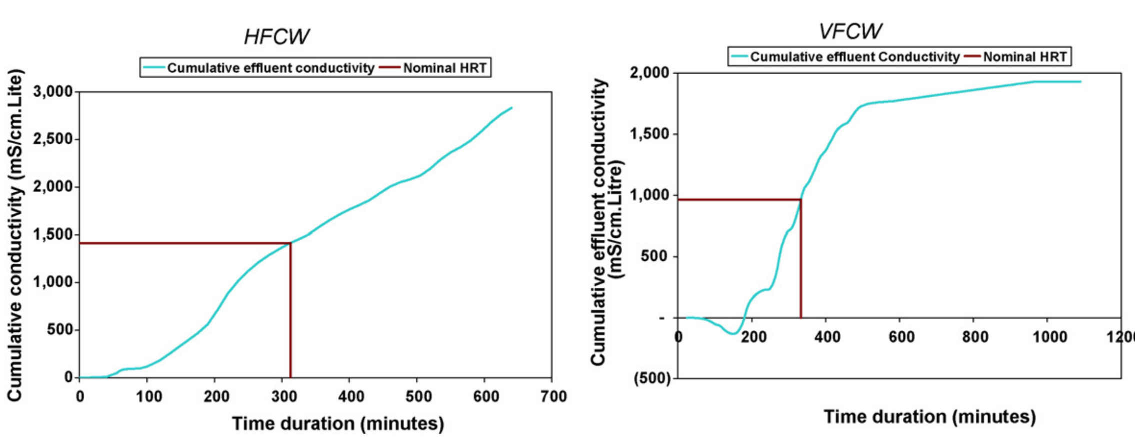

Fig. 2 – Cumulative effluent conductivity plotted against time.

Table 1 – Parameter concentrations.

Parameters	Units	Raw		ABR		HFCW		VFCW	
		Average	S.D.	Average	S.D.	Average	S.D.	Average	S.D.
TSS	mg/l	1506.3	1602.7	322.2	126.7	98.2	47.8	37.8	28.9
BOD_5	mg/l	1593.8	686.0	774.2	507.1	292.5	104.9	173.3	118.8
COD	mg/l	2914.2	1405.7	1421.9	923.3	647.3	395.0	318.6	235.9
NH_4–N	mg/l	142.0	20.2	209.3	103.7	150.2	49.3	45.0	42.0
TP	mg/l	24.4	7.6	28.4	9.6	18.5	5.6	17.1	7.1
FC	CFU/1 ml	7.5E + 05	1.0E + 06	1.1E + 06	8.7E + 05	2.5E + 05	4.1E + 05	6.1E + 03	5.0E + 03

Table 2 – Percentage removal efficiencies of parameters.

Parameters	ABR		HFCW		VFCW		Total	
	Average	S.D.	Average	S.D.	Average	S.D.	Average	S.D.
TSS	68.3	16.1	69.3	13.8	57.6	23.0	95.9	2.4
BOD_5	45.3	38.6	57.5	15.0	44.9	30.4	90.1	5.0
COD	47.2	26.1	51.4	25.8	45.7	34.4	90.0	5.7
NH_3–N	−47.5	68.8	23.8	20.2	70.9	21.5	69.5	25.4
TP	−30.5	73.6	27.3	41.8	0.0	41.5	26.1	33.6
FC	−268.8	409.0	68.8	40.6	73.7	36.7	97.5	3.0

compared to the conventional primary treatment septic tank. The average removal efficiency of TSS, BOD_5 and COD is 68%, 45.3% and 47.2% respectively (Table 2). The NH_4–N increase in the ABR could be due to the ammonification of the organic N, even though literature suggests that ammonification takes place more likely in aerobic conditions. In the anaerobic zone, where substrate (BOD_5) concentration is high, the absence of oxygen causes the microorganisms to release the stored intracellular polyphosphates by decomposition to simple orthophosphates. The decomposition of polyphosphate to orthophosphate results in an increase of soluble phosphorus, which could be a reason for the increase of TP concentration in the ABR. The high velocity of wastewater during the time of sampling, which could have displaced the settled microorganisms, might be a reason for the increase in the FC units in the ABR.

The average removal efficiencies in the HFCW of the parameters, TSS, BOD_5, COD, NH_4–N, TP and FC are 69%, 58%, 51%, 24%, 27% and 69%, respectively (Table 2). The removal efficiencies are lower than most of the literatures cite (Vymazal, 2002; Rousseau et al., 2004; Vymazal, 2005; Puigagut et al., 2007). However, it should be considered that the system under investigation is a hybrid system and VFCW would contribute to further elimination of the parameters. HFCW was designed to remove about 70% of the BOD_5 load.

The average removal efficiencies in the VFCW of the pollutants, TSS, BOD_5, COD, NH_4–N, TP and FC are 57%, 45%, 46%, 71%, 0% and 74%, respectively. The removal efficiencies are slightly lower than most of the literatures cite (Rousseau et al., 2004; Vymazal, 2005; Puigagut et al., 2007). The reason could be that the depth of the VFCW is less compared to the usual depth provided in the VFCW. This might have resulted in the lower Hydraulic Retention Time (HRT) and could be a possible reason for the below par performance of the VFCW.

The average FC removal in the DEWATS is 97.5%. Considering lower HRT in the system and decreased depth of the VFCW, the results are understandable.

Fig. 3 shows the pollutant concentrations at each stage of treatment throughout the period of research. It can be observed that removal efficiencies of the pollutants decreased in the winter period and increased in the summer period as expected since the pollutant removal is temperature dependent. There is a sudden increase in the removal efficiencies in July (except for FC) and this is due to the fact that the ABR was desludged in June 2007.

Table 3 shows the parameter concentration and efficiency in sludge treatment. The removal efficiency of the pollutants, TSS, BOD_5, COD, and TKN is 99.9%, 88.7%, 88.5% and 95.9%, respectively.

Table 3 – Parameter concentration and efficiency in sludge treatment.

Parameters	Sludge (mg/l)	SDB (mg/l)	VFCW (mg/l)	Removal efficiency (%)
TSS	100,550	984	74	99.9
BOD_5	3300	540	373	88.7
COD	7150	1375	820	88.5
TKN	280	6.5	11.5	95.9

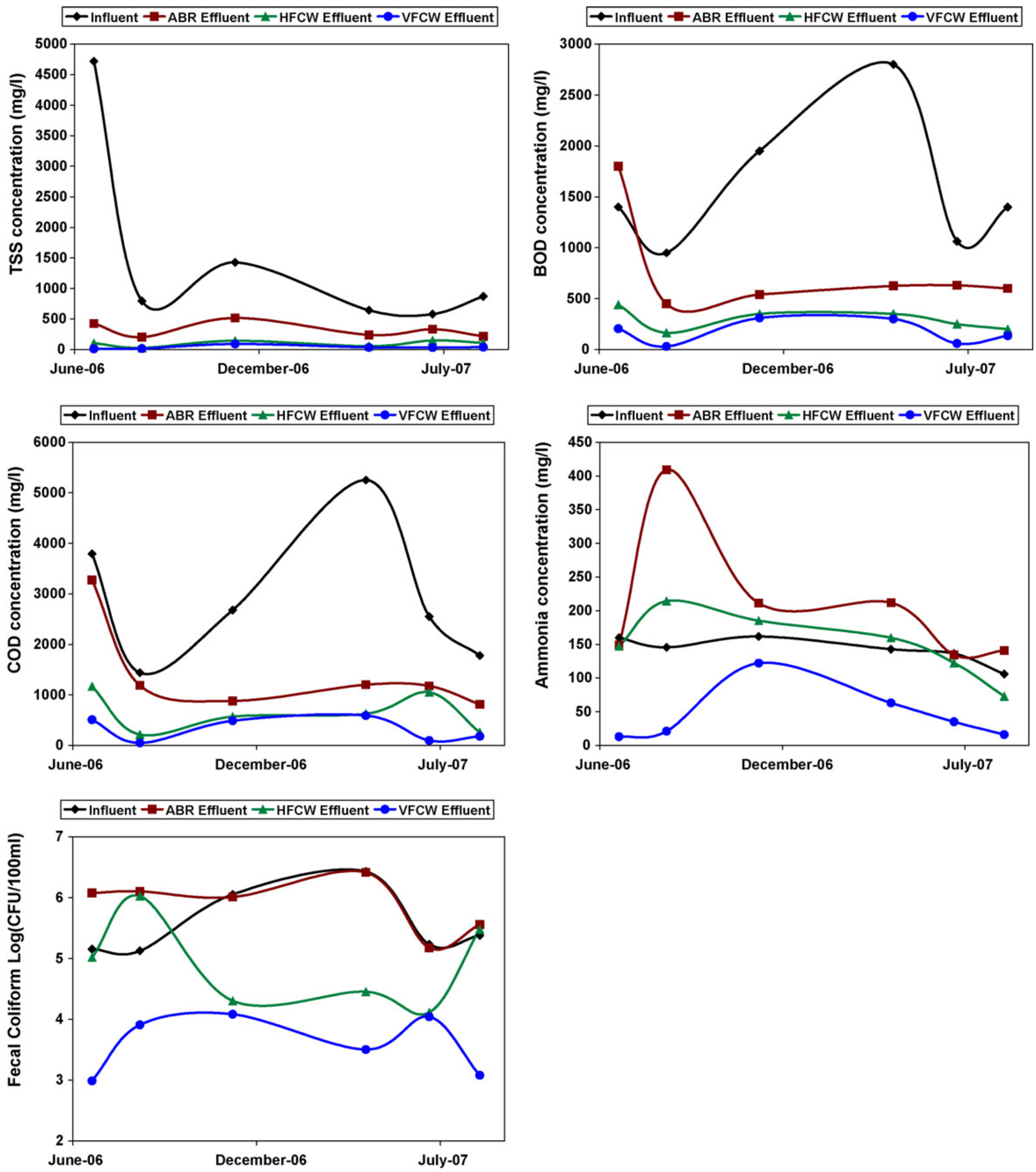

Fig. 3 – Parameter concentrations at each stage of the treatment.

4. Conclusions

It can be concluded that there is high potential of using ABR as primary treatment. ABR is very effective in the removal of organic parameters and could achieve TSS removal up to 91%, BOD up to 78% and COD up to 77%. The performance of the VFCW was not so encouraging because the shallower depth of 55 cm was used. The depth of the VFCW should be a minimum of 70 cm to achieve better performance in the removal of nutrients as well as organic pollutants (UN-HABITAT, 2008). With the total specific cost of DEWATS about US $ 80/PE (compared to US$ 800/PE – Rousseau et al., 2004; US$ 350/PE – Puigagut et al., 2007) and the annual O&M cost of about US $ 520 (compared to US$ 125/PE year – Puigagut et al., 2007). It can be concluded that it is one of the least cost option for DEWATS. In addition, it does not require skilled operators, which add up to the appropriateness of such DEWATS for developing countries like Nepal.

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