Development and adoption of wastewater treatment system for peri-urban agriculture in Multan, Pakistan

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ABSTRACT

The present research was conducted to assess the feasibility of biological treatment of a typical wastewater (WW) stream in Multan, Pakistan, using daily trends of WW characteristics and to design a wastewater treatment (WWT) system for that stream. The pH (5.8–6.2), temperature (24–30 °C), biological oxygen demand (BOD_5: 128–265 mg/L), ultimate BOD (BOD_u: 227–438 mg/L), BOD/total Kjeldahl nitrogen (BOD_5/TKN:5.9–11.2), BOD_u/BOD_5 (1.6–2.0), carbonaceous BOD_u/ nitrogenous BOD_u ($CBOD_u/NBOD_u$:1.6–2.8) of the WW was found to support the biological WWT. The inclusion of NBOD also indicated the need for nitrification-denitrification. The linear regression analysis of volatile suspended solids (VSS) with total suspended solids (TSS) indicated the high content of organic solids, which also made the WW suitable for biological treatment. The BOD/COD (chemical oxygen demand) <0.8 indicated the requirement for biomass acclimation. The major process units of the WWT system developed included a primary clarifier, cascade aeration, trickling filter, adsorption filter and chlorination contact tank. During the validation of design procedures, considerable removal of TSS (91%), TDS (46%), BOD_5 (88%), COD (87%) was observed over the 15 week operational period of the secondary WWT system. The WWT system developed was appropriate as a sustainable WWT system that consumed less energy and had lower operational costs.

Key words | biological treatment, design, diurnal variation, secondary treatment

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INTRODUCTION

The use of untreated wastewater (WW) for peri-urban agriculture is increasing greatly due to its consistent and cheap availability and to reduce urban poverty (Vazhacharickal & Gangopadhyay 2014). Farmers are induced to use WW for irrigation due to scattered agricultural land and the unavailability of conventional irrigation systems (Kannaujiya & Mishra 2018). However, using WW brings ecological and health hazards because of the high content of organic compounds, nutrients, various pathogenic organisms and toxic chemicals (Norton-Brandão *et al.* 2013). More than half of the world's water bodies and agricultural produce are seriously polluted due to untreated WW disposal/use. So, it should be treated appropriately before final reuse/ disposal (Ijaz *et al.* 2015; Margeta & Durin 2017). The ultimate objective of WW management is environmental protection in accordance with public health and socioeconomic aspects. This cannot be achieved without effective treatment of WW (Khan *et al.* 2015). So, WW treatment (WWT) is a complementary option in WW management. However WWT involves numerous competing physical, chemical and biological treatment options whose selection and design are important concerns for WW engineers. The cost and energy requirements are also challenging factors for WWT enhancement (Gikas 2017).

Numerous innovative sustainable technologies for WWT and resources recovery are available but knowledge concerning their selection and integration is limited (Khiewwijit *et al.* 2015; Mohammed & El Bably 2016). In order to establish preferred and sustainable WWT systems, the identification of pollutant dynamics and range of WW constituents is critical (Hamid *et al.* 2013; Droste & Gehr 2018). The treatment operations and processes needs to be optimized to mitigate the combinations of all expected contaminant ranges and flow levels. The aim is that, at all times, the effluent should meet established permissible limits for reuse objectives such as irrigation or industrial

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reuse, and surface or ground water body discharge. Thus integration of WWT technologies and understanding of influent variability and their treatment impact is essential to avoid environmental and socioeconomic impacts of reused WW. The relevant engineering factors, designated dimensions, bench and pilot scale studies are crucial for their effective operation.

Efficiency, sludge disposal features, reliability, environmental impacts, and land requirements are the critical parameters for the selection of wastewater treatment systems in developed countries. In developing countries like Pakistan, the construction and operational cost, power consumption, simplicity and sustainability are the critical aspects which are taken into consideration for development of WWT systems. These aspects can be achieved by customising the design of WWT systems according to the characteristics of existing WW streams, intended use of treated WW and receiving environmental conditions (Ali *et al.* 2017).

The oxidation pond treatment system provides good removal of biological oxygen demand (BOD) with zero power consumption and lower construction, process and maintenance costs as compared to other systems (Butler *et al.* 2017). However, its requirement for land and hydraulic retention time (HRT) is much higher. The land and cost requirements for an upflow anaerobic sludge blanket (UASB) reactor are less than all other systems except oxidation ponds and constructed wetlands (Gulhane & Pawar 2015). Activated sludge WWT is the most conventionally used technology in developed and industrialized countries for the removal of dissolved organics using the suspended growth processes. Fixed film or attached growth treatment systems have been used to overcome the hurdles of activated sludge technology. These hurdles are mostly related to poor settling of organic pollutants and increased costs of solids treatment (Naz *et al.* 2015). Biofilm treatment systems highlight the advantages of enhanced control of reaction kinetics, population dynamics, retention time reduction, operational flexibility, capacity improvement to decompose recalcitrant compounds and produce less sludge (Antonie 2018).

Trickling filter systems are thought to be quite effective among all biofilm treatment systems due to environmental compatibility, simple design, less biomass washout, operational ease, low energy costs and maintenance needs (Naz *et al.* 2015). It utilizes various contaminant removal mechanisms, including: biosorption, biodegradation, biomineralisation and bioaccumulation. These systems can be employed for the treatment of carbonaceous, nitrogenous and sulphurous matter from wastewaters. The trickling filter costs can be easily minimized using indigenous low-cost biofilm support media (Ali *et al.* 2016). Activated carbon based adsorption filter is not appropriate for developing countries due to its high cost and operational complications. Multilayer adsorption using indigenous natural sorption media can enhance the quality of secondary treated effluent as well as reduce the capital and operational cost (Zhang *et al.* 2015). There is a need to develop WWT system which can enhance the quality of treated effluent to full maturity for field applications with less power consumption and operational ease. Therefore, the present research study investigated the design, development and evaluation of an integrated pilot scale WWT system for one typical WW stream in Multan, Pakistan.

MATERIALS AND METHODS

Physico-chemical characterization

Daily wastewater characterization of influent WW was conducted for a week to assess its feasibility for biological treatment and to determine design loadings of the constituents of interest. The study was conducted at the disposal station of Bahauddin Zakariya University, Multan, Pakistan. Composite WW samples were collected every day for a week in a 1.5 L pre-sterilized bottle according to standard sampling procedures for validity of test results (APHA 2012). Samples were analysed immediately or stored at 4 °C with preservatives to delay biological and chemical changes. Physico-chemical analysis for pH (Senz pH Pro Trans EDT, (Sr. no. 1106-07863)), dissolved oxygen (DO; DO^{+6} EuTech (Sr. no. 662684)) and temperature was conducted on the spot using relevant meters for influent and effluent samples. The biodegradability indexes particulate BOD (BODp), soluble BOD (BODs), ultimate BOD (BOD_u), carbonaceous BOD_u ($CBOD_u$), nitrogenous BOD_u ($NBOD_u$), BOD_u/biological oxygen demand (BOD_5), $CBOD_u/NBOD_u$, and (chemical oxygen demand) COD/BOD were measured using standard procedures (5210, 5220 APHA 2012). BOD was determined by the 5-day BOD test (5210B) by dividing the difference in DO after 5 days of incubation to the dilution factor (APHA 2012). It serves to indicate waste loadings in a treatment system and the treatment efficiencies of such a treatment system. The COD of wastewater samples was determined by the closed reflux method (5220, APHA 2012) by measuring oxidation with a strong oxidizing agent such as potassium dichromate in the presence of sulphuric acid and silver after thermal digestion. Total dissolved solids (TDS) and total suspended solids (TSS) were measured by the gravimetric method (USEPA 8158, 8164, 8163) by filtration and oven drying the filter paper and residual samples, respectively. These results were used to assess the contamination strength, rate of biodegradation and design requirements (Lefkir *et al.* 2016). These constituents' loadings can be used for effective design of the treatment system, including TSS loadings for the clarifier design, BOD loadings to design biological treatment, nutrients such as total Kjeldahl nitrogen (TKN) for assessment of nitrification and inorganics for adsorption system. The selection of design loadings ($C \times Q$) of constituents of interest for respective treatment technologies was investigated after critical analysis of daily variations to simplify the design of the wastewater treatment system.

Design and development of pilot-scale WWT system

The wastewater treatment system was designed after selecting the design loading of WW constituents using standard design formulas and procedures (Table 1) (Metcalf & Eddy 2003). The different operations and processes of wastewater treatment were considered for the removal of a variety of contaminants such as solids, organics, nutrients, inorganics, heavy metals and pathogens (Buha *et al.* 2015; Mohammed & El Bably 2016; Seow *et al.* 2016). The developed wastewater treatment system consisted of physical, biological, adsorption and disinfection treatment segments. The flow rate estimation was conducted by the product of estimation of population, rate of water supply and percentage of water supply as sewage flow (Imam & Elnakar 2014). The primary clarifier was designed by estimating the settling characteristics of solids loading in terms of overflow rate (Nikolay 2017). Overflow rate was determined using the settling column test by measuring suspended solids at regular time intervals along the length of settling column (Metcalf & Eddy 2003). The area of clarification was estimated by the ratio of design flow to overflow rate. The retention time was estimated using the ratio of volume to design flow of clarifier. The cascade aeration system was designed by estimating the deficit ratio, wastewater temperature and parameters of water quality (a) and weir geometry (b) (Metcalf & Eddy 2003; Talib *et al.* 2010; Kumar *et al.* 2013; Kokila & Divya 2015). The deficit ratio was estimated using the DO concentration upstream (Co), downstream (C) and at saturation (Cs) (Baylar 2007; Kahil & Seif 2014). The parameters of water quality and weir geometry were estimated using Appendix D (Metcalf & Eddy 2003). The height of the cascade aeration system and other details were determined using a standard formula and Tables 5–34 (Metcalf & Eddy 2003). The design of the trickling filter was conducted using USA's National Research Council design equation by estimating the BOD loading (W), recirculation ratio (R) and removal efficiency (E) (Hicks 2000; Metcalf & Eddy 2003). The design BOD and hydraulic loading were estimated using the designed volume (V) determined by the NRC equation. The volume of upflow sequencing four carbon contactors for adsorption filtering of wastewater was investigated based on the research of different naturally occurring adsorbents for polishing of trickling filter effluent to enhance the removal of organics and inorganics (Cooney 1998; Metcalf & Eddy 2003; Zhang *et al.* 2015). Further design modifications of developed adsorption filters should be continued according to isotherm and kinetic analysis of adsorbents based on batch scale and column tests for design data modifications depending on the total surface area, bulk density, uniformity coefficient, porosity etc. of adsorbents used (Cooney 1998; Ali & Gupta 2006). The chlorination contact tank (CCT) for disinfection of pathogens was designed using dispersion considerations

Table 1 | Comprehensive approach towards WWT system design[a]

Sedimentation tank	Cascade aeration	Trickling/adsorption filter	Disinfection unit
Overflow rate estimation = Vo = Q/L × W Area = A = Q/Vo L × W or πd2/4 Retention time = tr = V/Q Depth = D = tr × Q/A	Deficit ratio = R = C_s – Co/ C_s – C Cascade height = H = R – 1/0.11 a b (1 + 0.046 T) Step length, height and inclination angle from Table 5–34 using 'H' (Metcalf & Eddy 2003)	BOD loading = W = Q × BOD_{inf} $F = 1 + R/(1 + R/10)^2$ Efficiency $E_{NRC} = 100/1 + 0.4432\sqrt{W/VF}$ Assume depth, find diameter $V = \frac{\pi D2 \times L}{4}$ adsorption Filter = $V/m = q_e / C_o\text{-}C_e$	CCT length = Q × contact time/ width × depth, V = Q/width × depth $N_R = 4VR/v_o$ R = A/P D = coefficient of dispersion = $1.01v_o\ (N_R)^{0.87}$ dispersion number = $D{\cdot}t/L^2$

[a]Details of design symbols used in formulas are provided in respective design section.

assuming the behaviour of plug flow reactors by axial dispersions (Metcalf & Eddy 2003). The baffle spacing was set at 10% of the whole CCT area. The treatment segments were designed to reduce WW impacts for peri-urban agriculture.

Experimental operation of secondary treatment system

Wastewater from the Agricultural Engineering department was fed into the main sewage line. Wastewater taken from the septic tank of the main sewage line of the Department of Agricultural Engineering was used to assess the removal performance of secondary treatment systems. A 1 HP submersible pump was installed in the septic tank of the sewage line to transfer wastewater from septic tank to the primary clarifier of the WWT system. The retention time of 45 minutes was given to the primary clarifier for the removal of suspended solids and particulate BOD. The primary treated wastewater was supplied to cascade aeration system mainly for DO enhancement by PVC pipes of 5 cm diameter. The primary treated and aerated wastewater was supplied to the tricking filter (Figure 1). To reduce the operational cost of the secondary treatment system, maize cob was used as the biofilm support medium for the trickling filter. The flow rate into the trickling filter was adjusted with the help of control valves fitted with pipe fittings. Here organic matter stabilization was accomplished by biological means using attached biofilm. This process produced protoplasm (biological floc) and various gases. The settling of protoplasm was accomplished in a secondary clarifier with a retention time of 60 min. Thus, the secondary clarified effluent was obtained at the outflow of secondary clarifier.

RESULTS AND DISCUSSION

Daily WW characterization

The characterization of WW was conducted to assess the daily variations, feasibility of biological treatment and design loadings. The observed daily variations of various wastewater constituents are shown in Figure 2 and Table 2. The pH variation (5.8–6.2) was almost constant to support the microorganism growth in WW treatment plant (Bai et al. 2011; Shah et al. 2014). This range also allows the agricultural reuse of the WW and helps metal fixing in the soil (Jiménez 2006). The temperature varied in the range of 24–30 °C, with standard deviation of 3.4, was within the required range of 10–40 to support biological treatment of WW (Grady et al. 2011). The TDS range 430–610 mg/L was suitable for agricultural use of WW (Pescod 1992). A linear regression analysis between TSS and volatile suspended solids (VSS) was VSS = 0.6414TSS – 10.016 with R^2 (0.94). A high content of organic solids was also found, which made the WW suitable for the biological treatment

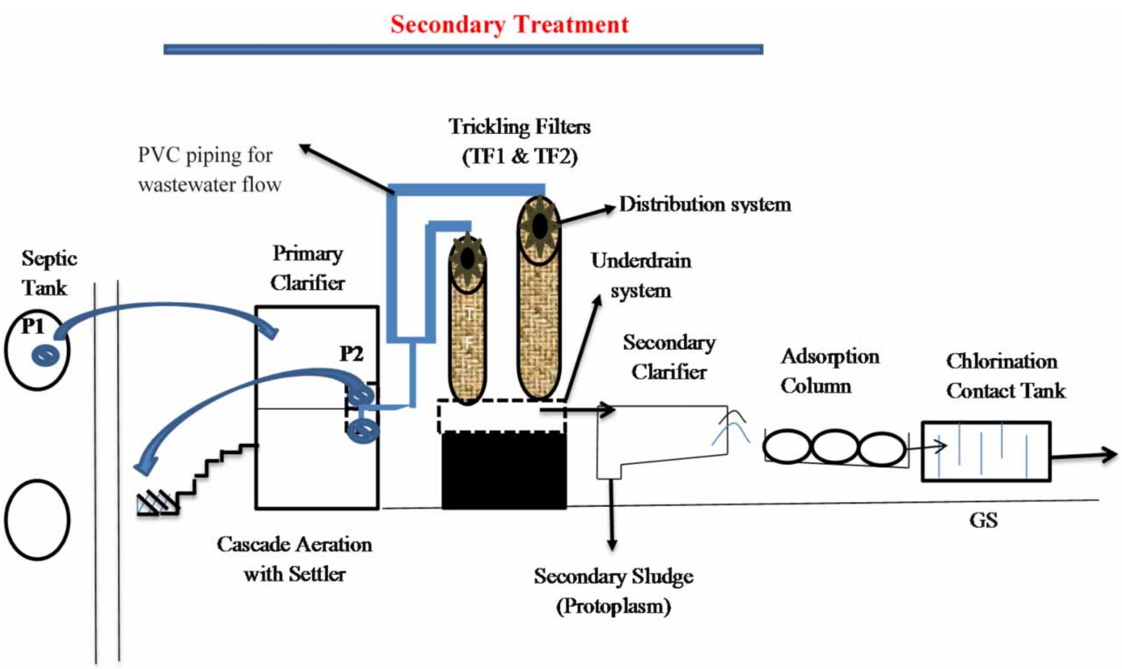

Figure 1 | Experiment set-up of the secondary WWT.

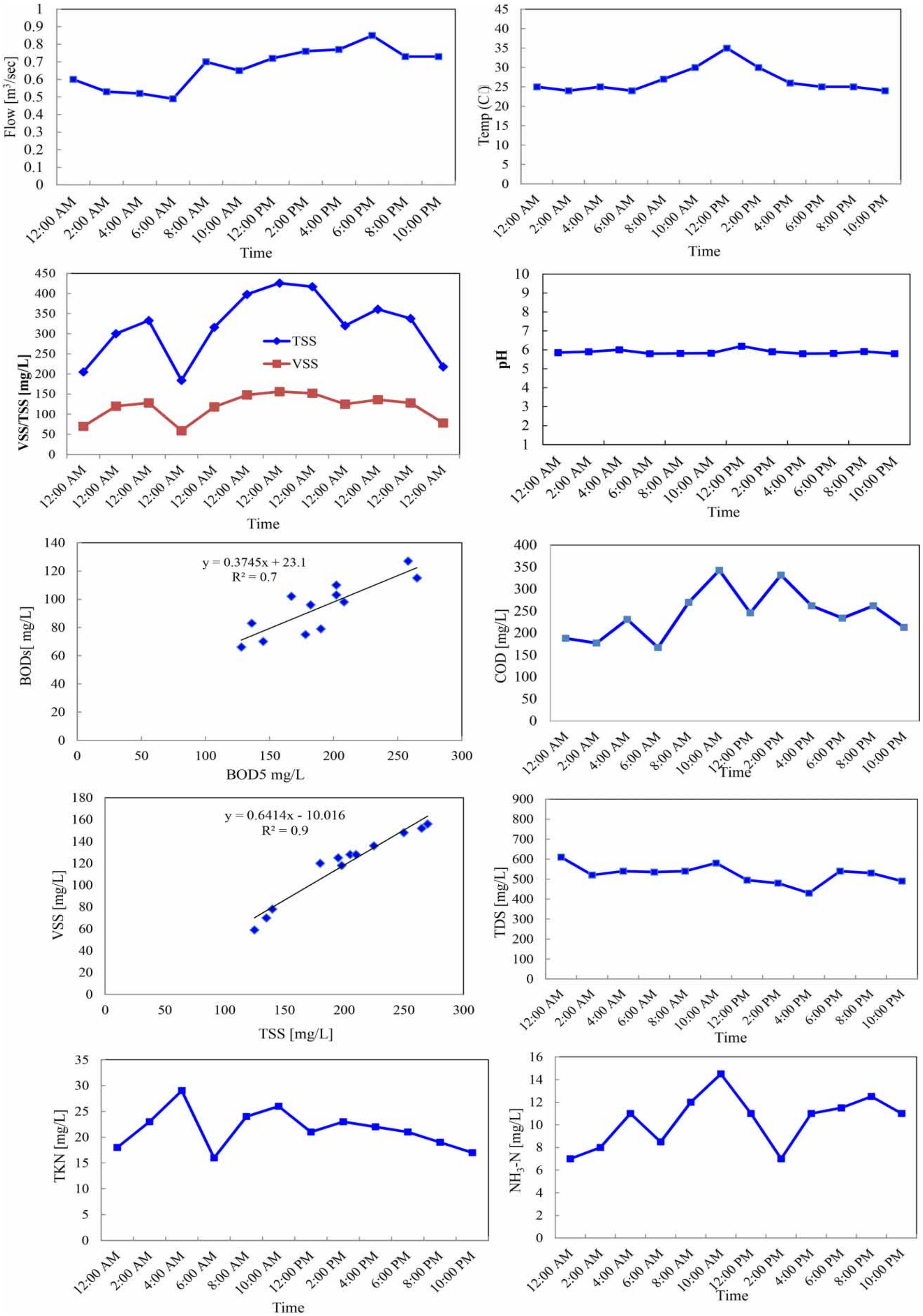

Figure 2 | Daily variations of WW constituents.

Table 2 | Diurnal variation results of oxygen demanding indicators

Time	BOD_5 (mg/L)	$CBOD_u$ (mg/L)	$NBOD_u$ (mg/L)	BOD_u (mg/L)	BOD_u/BOD_5	$CBOD_u/NBOD_u$	BOD_5/TKN
12:00AM	145	174.7	82.3	257.0	1.8	2.1	8.1
2:00 AM	136	163.9	105.1	269.0	2.0	1.6	5.9
4:00 AM	178	214.5	132.5	347.0	1.9	1.6	6.1
6:00 AM	128	154.2	73.1	227.3	1.8	2.1	8.0
8:00 AM	208	250.6	109.7	360.3	1.7	2.3	8.7
10:00 AM	265	319.3	118.8	438.1	1.7	2.7	10.2
12:00 PM	190	228.9	96.0	324.9	1.7	2.4	9.0
2:00 PM	258	310.8	105.1	416.0	1.6	3.0	11.2
4:00 PM	202	243.4	100.5	343.9	1.7	2.4	9.2
6:00 PM	182	219.3	96.0	315.2	1.7	2.3	8.7
8:00 PM	202	243.4	86.8	330.2	1.6	2.8	10.6
10:00 PM	167	201.2	77.7	278.9	1.7	2.6	9.8

(Hamoda *et al.* 2004). The parameter TKN is used for rivers DO modelling, especially when modelling involves NBOD with CBOD (Haider & Ali 2016a). The estimation of $NBOD_u$ is important for impact assessment on the receiving environment where sufficient nitrifiers are available to carry out NBOD (Haider & Ali 2016a, 2016b). TKN and NH_3-N were 21.6 mg/L and 10.4 mg/L, with standard deviations of 3.7 and 2.3 respectively. DO is the indicator of river water quality health and BOD is the supreme monitoring parameter for DO in receiving water bodies (Akoteyon *et al.* 2011). In this study, tests for both DO and BOD_5 were performed, giving average values of 94 mg/L and 188 mg/L, with standard deviations of 19 and 43 respectively. The BOD and COD ratio of influent was observed to be less than 0.8, which indicates a requirement for pre-treatment and biomass acclimation (Abdalla & Hammam 2014; Aslam *et al.* 2017). For assessment of the WW impact on the receiving environment, determination of oxygen demanding indicators such as BOD_5, $CBOD_U$, $NBOD_U$ and BOD_U is essential to investigate the complete degradation potential of the WW. The CBOD bottle rate constant was 0.35/day. Their estimation is given in Table 2.

$$CBOD_U = BOD_5/1\text{-}e^{-kt}$$

$$NBOD_U = 4.575TKN$$

$$BOD_U = CBOD_U + NBOD_U$$

The average values of BOD_U, $CBOD_U$ and $NBOD_U$ were 326 mg/L, 227 mg/L and 99 mg/L with standard deviation of 62.3, 17.3 and 51.8 (Table 2). The minimum BOD_U was found at 6:00 AM while the maximum BOD_U value was found at 10:00 AM. It was observed that BOD_U was more than twice BOD_5, due to the addition of NBOD. The $CBOD_U/NBOD_U$ ratio was in the range of 1.6–3. Less difference between $CBOD_U$ and $NBOD_U$ was observed at night while a big difference was observed in the day due the interference of industrial and laboratory activities. The inclusion of NBOD reveals that the design of biological treatment for Bahauddin Zakariya University WW should be focused on both BOD removal and nitrification-denitrification (Tanner *et al.* 2002).

Specifications of the WW treatment system developed

The design of the primary clarifier, cascade aeration system, trickling filter, secondary clarifier, adsorption filter and chlorination tank was completed using the guidelines developed by Metcalf & Eddy 2003 (Figure 3). Many researchers have used these guidelines for designing WWT systems. So, the present WW treatment system was also critically designed using some solid assumptions and reliable WW loading data. Thus design specifications of selected technologies have been calculated (Table 3). Some assumptions were made during the design of the WWT system, but the recommendation is to reduce these assumptions as much as possible to achieve the most accurate and reliable results.

Primary clarifier design has generally been done using more empirical than rational approaches due to the lack of understanding of which pollutants it is possible to remove. Normally, it is stated by wastewater treatment plant engineers that the primary clarifiers are designed for the removal of 60% of the TSS. Actuality, the primary

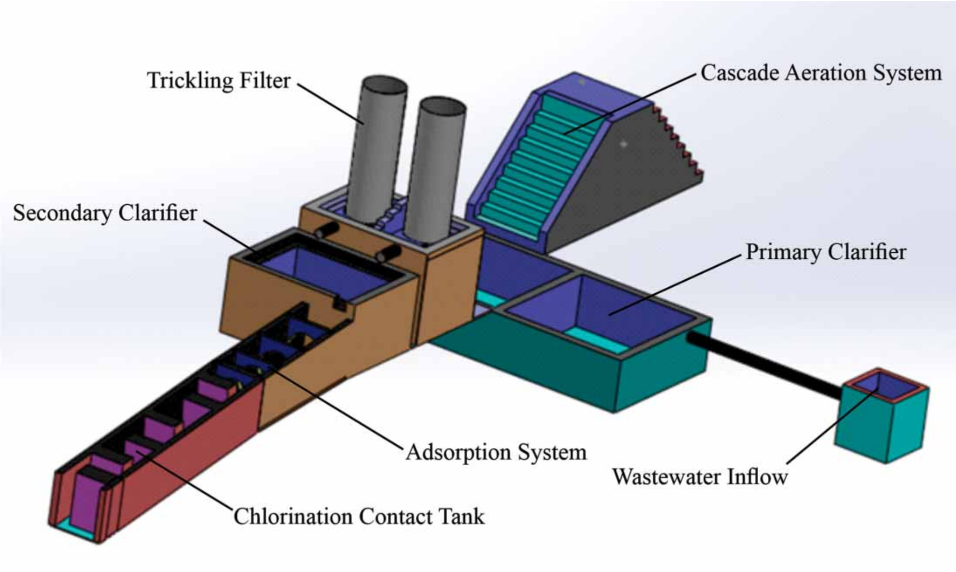

Figure 3 | Developed wastewater treatment system.

clarifiers are not designed for 60% removal but are assumed to do so (Nikolay 2017). In the present research study, the design of the primary clarifiers was based on the removal of completely settleable TSS during flow conditions of average dry weather. The concentration of settleable TSS is a wastewater characteristic that can be easily assessed by the settling column test. So wastewater characterization was utilized for good design of the primary clarifier. A rectangular tank of 3.1 m × 3.1 m and 1.5 m depth was constructed for primary settling of suspended organic and inorganic constituents of WW. However, specific HRTs were selected after settling trials with the particular WW stream. The present practice of primary sedimentation design was based on the overflow rate (Table 4). The limited sedimentation operation was designed and developed depending upon the WW characteristics to reduce TSS and BOD_P to a desired value using determined HRTs (Topare *et al.* 2011). The sedimentation tank is about a quarter of the capital investment of a treatment system. In order to evaluate the effectiveness of this treatment (sedimentation), the TSS and BOD should be monitored before and after treatment.

The constructed cascade aeration system enhanced the DO and efficiency of the primary WWT and avoided organic overloading of secondary treatment. The stepped cascades were mainly focused on the design of a stepped spillway, pre-aeration or post aeration WWT. The stepped spillway

Table 3 | Design specifications of WW treatment technologies[a]

Technology	Length	Width	Depth	Diameter	Slope	Specifications
Primary clarifier	3.1	3.1	1.25		Towards sludge hopper 0.3	Sludge hopper (0.3 × 0.3)
Cascade aeration	1.03	2.1	2.6		Inclination angle 40°	Steps (S) Total S: 11 SH: 0.75 SW: 1.08
Settler	3.1	3.1	1.25		Towards sludge hopper 0.3	Sludge hopper (0.3 × 0.3)
Trickling filter-1			2.3	1.00		Depth of filter media: 1.93
Trickling filter-2			2.9	1.00		Depth of filter media: 2.07
Secondary clarifier	2.70	2.13	1.52			Sludge hopper 0.3 × 0.6
Adsorption filter	3.40	1.3	1.2–0.3		0.24	4 portions for different adsorbents
Chlorination tank	3.80	1.28	0.85–0.6		0.24	Baffles No. 10 BH: 0.5

[a]All dimensions are in metres.

Table 4 | Design of the sedimentation tank

Average discharge = population × rate of water supply × % of water supply as sewage flow = 4,500 × 0.03 m^3/no./day × 0.75 = 100 m^3/day
Retention time = 3 hours
Capacity of sedimentation tank = 100 × 3/24 = 12 m^3 = L × b × d = 3.1 × 3.1 × 1.25
➔ Overflow rate = 10.4 m^3/m^2·day and A = Q/Vo = 100/10.5 = 9.6 m^2 (3.1 × 3.1)
t_r = V/Q = AxD/Q ➔ D = t_r x Q/A = 1.25 m

design approach is considered to be an energy dissipation component. The process of pre-aeration is used to remove manganese and iron salts in the water, but post aeration encourages high levels of DO (5–8 mg/L) in the effluent to meet the standards (Metcalf & Eddy 2003). The cascade aerator can enhance the treatment efficiency of the trickling filter. Non-dimensional parameters such as cascade steps, ratio of cascade height to bottom radius etc. were proposed accordingly. Kumar *et al.* (2013) also used a similar approach for the design of a pooled circular cascade aeration system. The total height of the cascade aeration structure was about 2.6 m using the of characteristics of the raw WW and the desired level of DO. The individual step height and step width were about 0.22 m and 0.3 m respectively (Table 5). An inclination angle of about 40° was considered sufficient for aeration purposes. The cascade aeration system was found to be a low cost method for aeration of raw domestic or primary treated effluents to further reduce the biodegradable organics or BOD (in terms of COD) as required (Kumar *et al.* 2013; Kahil & Seif 2014). Several water quality monitoring reports and research studies have pointed out the depleted DO level in surface water bodies of Pakistan. The major cause of DO depletion is the disposal of untreated wastewater. The principle of aeration is based on air being trap into the WW due to oxygen diffusion/transfer from the atmosphere based on the turbulence of WW flowing over a series of cascade steps (Moulick *et al.* 2010; Kokila & Divya 2015). The system efficiency should be evaluated in terms of COD, BOD, TSS and TN before and after the treatment process. After limited sedimentation operation and cascade aeration, the physical and chemical characteristics of WW should be determined. If the characteristics of WW are within the permissible limit then it can be recommended for irrigation, otherwise further processing of WW should be carried out by the trickling filter and adsorption filter.

Table 5 | Designing procedure of cascade aeration system

Cascade aeration = H = R–1/0.11 a b (1 + 0.046 T),
R = deficit ratio = Cs-Co/Cs-C
Cs = a DO conc. at temp. T (mg/L)
Co = DO conc. of influent (mg/L) = 1 mg/L
C = required final DO level after post aeration (mg/L) = 6 mg/L
Determine the 'a' (water quality parameter) -, 'b' (weir geometry parameter (for steps)) and Cs values from (Metcalf & Eddy 2003) Appendix D and T selected was 20 °C
So, cascade height for 20 °C = H = 2.7 m and
Cascade height for 25 °C = H = 3.3 m
If height is 1.8–2.4 m then from Table 5–34 (Metcalf & Eddy 2003) step height is 22.3 cm, step length is 33 cm and inclination angle is 40°
Thus, we use height of 2.6 m with 11 steps and width of 0.22 m and 0.3 m.

The developed NRC model based on the volumetric loading and BOD kinetics dictated the trickling filter performance (Albertson 1989). So, the NRC model can be modified for any biofilm support media. It was used for the maize cob biofilm support medium for trickling filter application. The depth to diameter ratio of the trickling filter was more than the usual to enhance nitrification and denitrification rates. The secondary treatment of WW was accomplished using trickling filter treatment technology. Two-stage trickling filters of stainless steel as a cylindrical barrel with diameter 1 m and depth of 2.3 m (TF1) and 2.9 m (TF2) were installed using the NRC model (Table 6). Such dimensions were appropriate for applications in small farming communities to reuse WW for irrigation purposes. However, the depth of biofilm support was selected according to the filter media and the WW strength/pollution

Table 6 | Comprehensive design of trickling filters

Influent BOD = 150 mg/L
Design flow = 50 m^3/day
Recirculation ratio = 0.5:1
BOD removal efficiency in primary sedimentation = 30%
Recirculation factor = F = 1 + R/(1 + R/10)^2
Primary effluent BOD = (1–0.3) × 150 = 105 mg/L
BOD loading = W = 105 × 50 × 1/1,000 = 5.2 kg BOD/day
Removal efficiency = E = 65%
$E = 100/1 + 0.4432\sqrt{W/VF}$
By putting all values, we get V = 2.3 m^3 A = V/D
BOD loading = 5.2/2.3 = 2.2 kg BOD/m^3·d
Hydraulic loading = (1 + 0.5) × 50/1440 × 0.8 = 0.065 m^3/m^2·min
Assume, depth of TF = 2.9 m
$V = \frac{\pi D2 \ \times L}{4}$ ➔ Diameter = 1 m
So, to cover the whole flow of 100 m^3/d: two TFs have been installed
TF1 = dia. 1 m with depth 2.9 m, TF2 = dia. 1 m with depth 2.3 m

load. A trickling filter was investigated as an attractive secondary biological WW treatment option, being simple and reliable for the treatment of dissolved organics from WW (Ali *et al.* 2016). It was also found to be an appropriate technology for small to medium sized communities and peri-urban agriculture due to the lower space requirement and operational complications. Trickling filters are resilient against power failures and shock loads (Naz *et al.* 2016; Gikas 2017). However, some problems were observed related to the cost and operation of trickling filters which need to be resolved (Ali *et al.* 2017). However, appropriate modifications in design and construction of trickling filters should continued to render them suitable for adoption in Pakistan.

The operation of trickling filters is based on the trickling of WW over a biofilm support medium. The WW flows downward over the support medium surfaces from the sprinkling distribution arm. The bacteria/microorganisms in the influent WW attach themselves to the support medium and, in fact, a biofilm/slime layer starts developing on the filter medium surfaces. The biofilm grows in thickness and eventually the microorganisms residing in the biofilm react with the passing WW. In short, the WW gets treated as the biodegradable organics and the oxygen are transferred from the WW to the biofilm through diffusion and sorption (Eding *et al.* 2006). The organic pollutants get degraded (biological floc) and the water gets purified during its passage from the top of the filter to the bottom. The settling of biological flocs is accomplished in secondary clarifier.

Four carbon contactors of total length 3.4 m, width 1.3 m and depth 1.2–0.3 m were constructed for the effective removal non-biodegradable contaminants. The down flow design of the carbon contactors was to accomplish the adsorption of inorganics and the filtration of suspended solids in a single step to lessen the chance of accumulating particulates in the bottom of bed. The down flow adsorption system was developed to enhance multilayer adsorption by natural sorption media (Zhang *et al.* 2015). The further design modifications of developed adsorption filter should be continued according to isotherm and kinetic analysis of adsorbents based on batch scale and column tests for design data modifications depending on the total surface area, bulk density, uniformity coefficient, porosity etc. of adsorbents used by following conventional design equations for adsorption filters. The design of adsorption carbon contactors was based on four factors: contact time, hydraulic loading rate, carbon depth and number of contactors. In addition to the biodegradable pollutants, it was also necessary to remove non-biodegradable constituents, particularly the nitrogen, phosphorus and heavy metals from WW for peri-urban agriculture (Salem *et al.* 2014). Depending upon the presence of specific pollutants, adsorption filters are generally used for removing non-biodegradable contaminants from the WW as against the tricking filters, which are used for removal of biodegradable pollutants. The adsorption filters are simple and easy to construct and they are normally economical in design and operation.

Based on the most probable number (MPN) of pathogens indicators, such as total coliform and *Escherichia coli*, a CCT of length 3.8 m, width 1.28 m, depth 0.85 m–0.6 m was designed and developed to ensure effective disinfection of WW (Metcalf & Eddy 2003) (Table 7). The CCT was designed as a rectangular channel, with 10 baffles to prevent short-circuiting and to provide a specific contact time of about 30 min. The baffle spacing was set at 10% of the whole CCT area (Metcalf & Eddy 2003). The main objective of a CCT is to ensure that some defined percentage of the flow remains in contact with disinfectants for the designed contact time to ensure effective disinfection (Mezzanotte *et al.* 2007). The design of a CCT is normally based on the assumption of plug flow: the fluid is evenly distributed over the entire basin and retained for a period known as theoretical detention time. However, fluid particles entering all at once are observed to have unequal passage times and a significant proportion of fluids do not follow the theoretical detention time. So chlorine dosage must be increased to enhance the effectiveness of chlorination (Yang *et al.* 2017). But this practice increases the operational cost of disinfection. Therefore the present CCT was developed based on an actual experiment of dispersion of fluid in the CCT basin.

Performance of the secondary treatment segment of the WW treatment system developed

The efficiency of the WW treatment was investigated to develop a secondary WW treatment system that included a primary clarifier, cascade aeration system and trickling filter with maize cob biofilm support medium at temperatures of 23–37 °C and flow of 2.3 L/s for TSS, TDS, BOD and COD removal for the validity of the adopted design procedures.

The pH variation in the influent and effluent was in the range of 6.2–8.3 for the operational period of 15 weeks. It was found that WW having an acidic range of pH significantly decreased the oxidation rate of ammonium. So, the maintenance of a 6.5–8.5 pH range is required to optimize

Table 7 | Design considerations of CCT

First we determine the capacity of chlorinator with flow 25,000 gal/day → 100.3 m^3/day

Assume peak daily factor for treatment plant was 3 and maximum required chlorine dosage set by state regulation is 20 mg/L

Cl_2 kg/day = 20 g/m^3 × 100.3 m^3/day × 1 kg/10^3g

= 2 kg/day

Although the peak capacity will not be required during most of the day, it must be available to meet the chlorine requirements at peak flow. So, estimation of daily consumption of chlorine at average dosage of 10 mg/L

Cl_2 kg/day = 10 g/m^3 × 100.2 m^3/day × 1 kg/10^3g

= 1 kg/day

Chlorine contact tank design is as follows:

Assume some trial cross sectional dimensions for chlorine contact tank based on design flow and chlorine dosage:

Width = 4.2 ft (1.28 m), depth = 2.8 ft (0.85 m)

Desired dispersion number = 0.15

Detention time at peak flow of 100.2 m^3/day is 60 min

So length of CCT = Q × contact time/width × depth

= 100.2 m^3/d × 60 min/1440 min/d × 1.28 × 0.85

= 3.8 m (12.5 ft)

Check velocity at peak flow

V = Q/width × depth = 100.2 m^3 /d/1440 min/d × 60 sec/min (1.28 × 0.85)

= 0.0011 m/sec

Computing the dispersion number a:

Reynolds number = $N_R = 4VR/v_o$

V = 0.0011, R = A/P = 1.28 × 0.85/2(1.28 + 0.85) = 0.25

v_o = velocity in open channel = 1.003×10^{-6}

By putting values, we get N_R = 1120

D = coefficient of dispersion = $1.01 v_o \, (N_R)^{0.87} = 4.55 \times 10^{-4}$

Dispersion number = $D \cdot t/L^2 = 4.55 \times 10^{-4}$ (60 min × 60 min/sec)/$(3.8)^2$ = 0.11 < 0.15

Because the computed dispersion number was below the desired dispersion number, the given design was valid for chlorination

the treatment system performance (WHO 2006; US-EPA 2007). The results of present study were also in that range, indicating the effectiveness of the developed secondary treatment system.

TSS and TDS removal from the influent was about 91% and 46%. TSS and TDS concentrations in the treated effluent were recorded as 16 ± 13 and 263 ± 40 (Figure 4(a) and 4(b)). The higher TSS removal indicated the validity of the overflow rate calculated for clarifier design. The variation in TDS and TSS removal efficiency that was observed may have been due to the sloughing off of accumulated materials or the degradation of the biofilm support medium during trickling filter operation (Aslam *et al.* 2017). Rasool *et al.* (2018) reported TDS and TSS removals of 62.8% and 99.9% respectively from secondary treatment of WW using a trickling filter. Naz *et al.* (2015) reported TSS removal efficiency as 90.1, 77.2, 60.8 and 55.5% for biofilm support media of stone, plastic, polystyrene and rubber, respectively, in trickling filters.

BOD and COD are commonly used to determine the contamination strength and rate of biodegradation in WW. BOD and COD removal from effluent were 88% and 87% respectively, with DO enhancement of 0.2 to 6.51 mg/L (Figure 4(c) and (d)). DO enhancement with decrease in BOD and COD indicate active metabolism by microbes attached to the biofilm. Similar positive correlations between BOD and COD removal with DO enhancement have been reported by various researchers for secondary treatment using trickling filters (Sa & Boaventura 2001; Ali *et al.* 2016; Aslam *et al.* 2017; Rasool *et al.* 2018).

The power consumption of developed wastewater treatment system was estimated as 0.4–0.7 kWh/m^3, considerably less than for aerobic (2 kWh/m^3) and anaerobic (0.03–3.572 kWh/m^3) membrane bioreactors (Martin *et al.* 2011).

Research on developed WW treatment systems should be continued for other constituents of interest for WW. The developed and installed WWT systems should be tested and evaluated for enhanced removal of solids, BOD, COD, nutrients, heavy metals and pathogens. The adsorption columns of developed WW treatment systems should be further evaluated for nitrogen, phosphorus and heavy metal removal using naturally occurring adsorbents such as laterite, andesite, refuse concrete, limestone, rice husks etc. Similarly, CCTs should be investigated for MPN removal of pathogen indicators using various disinfectants such as sodium hypochlorite, hydrogen peroxide, chlorine compounds etc. Different treatment process models should

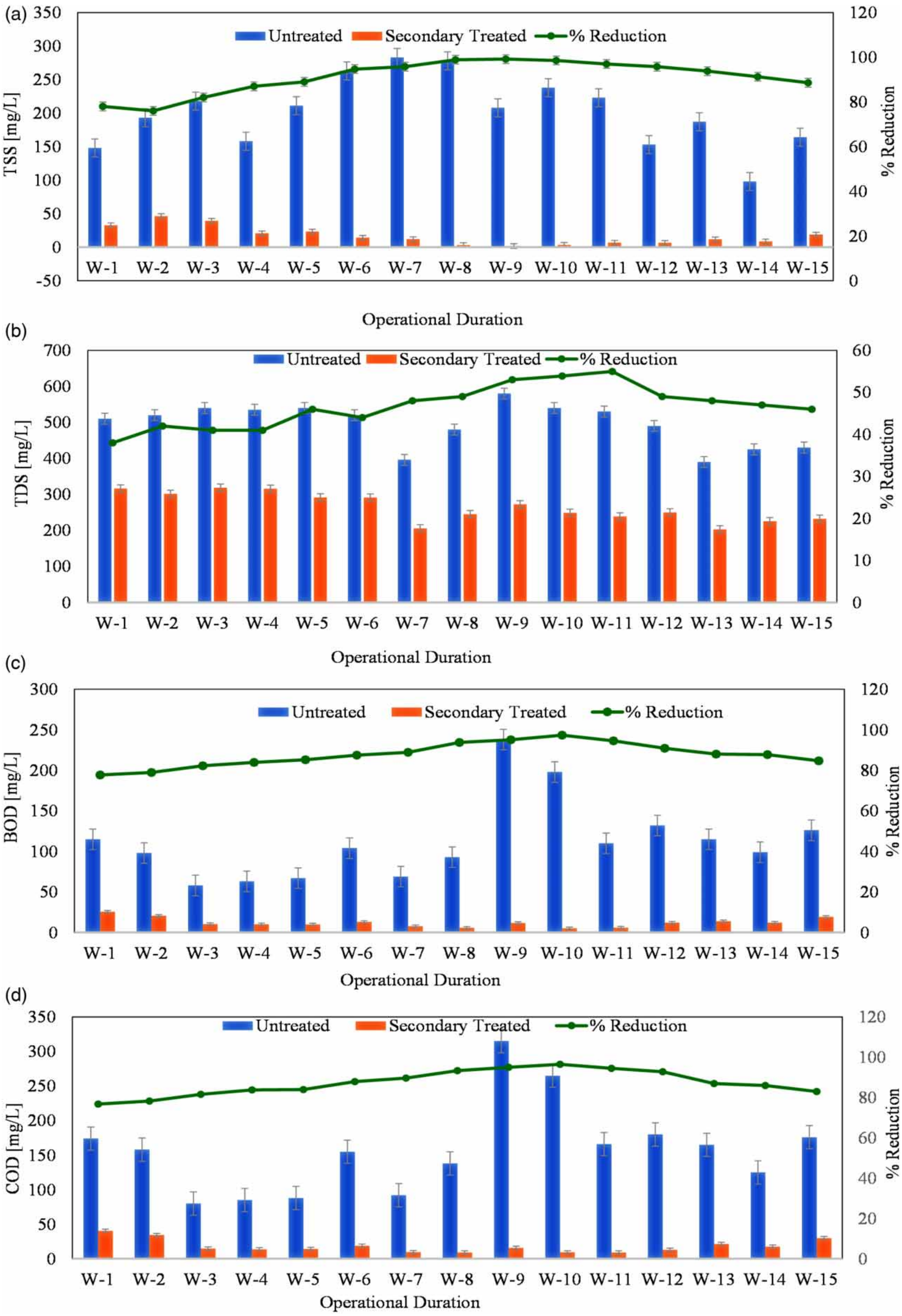

Figure 4 | Removal of TSS, TDS, BOD and COD from the secondary treatment system developed.

be used to investigate the treatment behaviour of developed wastewater treatment systems.

CONCLUSIONS

The feasibility of biological treatment of WW was concluded on the basis of pH (5.8–6.2), temperature (24–30 °C), BOD (128–265 mg/L), BOD_U (227–438 mg/L), BOD_5/TKN (5.9–11.2), BOD_U/BOD_5 (1.6–2.0), $CBOD_U/NBOD_U$ (1.6–2.8). They were also used to indicate the degradation potential of WW constituents by biological means. The present approach was effective for the proper selection, design, integration and development of WW treatment technologies using design loadings determined from daily WW constituent variations. This approach was found to be helpful to avoid oversizing and under sizing of treatment projects and treatment plant imperfections. The secondary WW treatment system was found effective for the removal of TSS (91%), TDS (46%), BOD5 (88%), COD (87%). The validity of design procedures and assumptions was concluded on the basis of efficient removal of the secondary WW treatment system. The WW treatment system developed was less energy demanding, easy to operate and potentially applicable for developing countries. The adsorption and disinfection units of developed WW treatment system should be further evaluated for added removal of organics, inorganics and pathogens.

DISCLOSURE STATEMENT

No potential conflict of interest was reported by the authors.

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