Decentralized treatment and recycling of greywater from a school in rural India

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Abstract

Rural areas in developing countries face the twin challenges of water scarcity and risk of groundwater contamination due to lack of water treatment options. A decentralized greywater treatment system for reuse is an option that addresses both of these challenges. This study reports the performance of a decentralized greywater treatment and reuse system which was constructed and operated for over 12 months in a government-managed school in rural India. The handwash and kitchen wash wastewater streams were treated separately due to differences in the initial greywater characteristics. The treatment stages included pre-treatment using screens and grease traps, slow sand biofiltration combined with anaerobic sludge bioreactor, and aeration before the final ozone-based disinfection stage. The treated water at the end of all these stages was used for toilet-flushing in the school. The treatment system was operated for one year and sampling was performed to investigate the system performance. The overall treatment system showed removal efficiencies of 99%, 98%, 66%, 73%, 98%, 96% and >99.99% for the parameters of turbidity, total suspended solids, nitrate, total phosphorus, biological oxygen demand (5 days), chemical oxygen demand and fecal coliform respectively. This study quantifies the performance of each subsystem and demonstrates for the first time that a decentralized greywater treatment can be operated effectively and economically in a rural Indian setting.

1. Introduction

With increasing population, climate change and expanding pressures on water resources, much of the world faces a major water crisis. Globally, water shortages are estimated to affect more than 4 billion people annually [1]. India occupies only 2.4% of the world’s total land area yet supports over 17.5% of the global population [2]. The total freshwater resource of the country is only 4% of the world’s total utilisable water resource [3], which is disproportionately low for the current population. In India, over 600 million people face high to extreme water scarcity, with water contamination estimated to impact as much as 70% of the country’s utilisable water resource [4]. Currently, the disparity between water supply and demand is widening due to increasing water scarcity [5,6], population growth, contamination of available surface water sources and depleting groundwater reserves. As this disparity worsens, there is a growing need for technologies that can address the interactions between poor water quality and insufficient water quantity [7,8]. Despite the extensive scientific and technological advances, the discharge of untreated wastewater (WW) in India still poses environmental and human health risks [9]. Treatment technology exists, but today these technologies are based on the conventional large-scale centralized WW treatment plants, where WW is collected from various sources and brought to a centralized WW treatment plant through extensive pipe networks [10]. Decentralized, efficient, on-site treatment and reuse of WW in general and greywater in particular, has the potential to realize the dual benefits of reducing consumption of freshwater and sustainably managing WW, especially in rural and peri-urban areas [11]. Though the reuse of greywater has a lot of...
potential, there are obstacles to its reuse, including but not limited
topublic health concerns and human perceptions of using treated water
[12].

Wastewater generated from households typically consists of black-
water (BW) and greywater (GW). BW is defined as wastewater produced
in toilets, whereas freshwater soiled by use in laundry, baths, showers,
hand washbasins, dishwashers, and kitchen sinks is called GW [13].
Contaminants present in the GW includes oil, food waste from kitchen
water and surfactants from all household cleaning and personal care
products. Relative to BW [14], GW has characteristically low suspended
solids [15], pathogens and nitrogen [16]. The quality of supplied
freshwater and the type of water distribution system (continuous vs
intermittent supply) is known to affect the composition of GW [17]. Due
to these characteristics, GW represents a huge potential for domestic
water savings through reuse. In many parts of the world, GW is reused
defined. The treated GW was recycled and its effect on the school
holders in a demand-driven approach. The treatment scheme used
operation and maintenance of such systems, but also because of
social sensitivities that surround human interactions with waste-
water [32]. Realizing the full potential of GW reuse requires
treatment plants but also captures and compares
the social sensitivities that surround human interactions with waste-
water [32]. Realizing the full potential of GW reuse requires
cost-effective, proven and efficient approaches to treatment that are
adaptable to site-specific hydro-social conditions [33].

This work reports the design and performance of a decentralized,
gravity-driven GW treatment and reuse system designed by integrating
different technologies specifically for a government school in rural
India. The GW recycling system was co-designed by engaging stake-
holders in a demand-driven approach. The treatment scheme used
disinfection techniques such as ozonation by installing ozoneators spe-
cifically designed for the rural Indian setting. This was done to ensure
the system was not reliant on disinfection chemicals, such as chlorine,
which need to be procured from a nearby city or town. This study is
unique because not only does it demonstrate the successful functioning
of a decentralized GW treatment plant but also captures and compares
the performance of various treatment options. The system was operated
for over 12 months by students and staff in a rural government school.
The performance of each of the different treatment modules is quanti-
fied. The treated GW was recycled and its effect on the school’s annual
water budget is reported.

2. Materials and methods

2.1. Study location and size

This study was conducted in the Berambadi Primary School
(11°45′ 44″ N, 76°34′ 03″ E) located in Berambadi village (Population of
2982 as of 2011) [34], Chamrajannerud district in the Indian state of
Karnataka. The school is located in the Berambadi watershed
(11°43′ 00″ – 11°46′ 00″ N, 76°31′ 00″ – 76°40′ 00″ E), which is clas-
sified as AW (Tropical wet and dry or Savannah Climate) based on the
revised Köppen – Geiger climate classification. The area receives an
average annual rainfall of 1000 mm [35].

Typically, schools do not generate as much per capita GW as do-
main households, owing to the absence of GW sources such as laundry
and showers. The government-run schools in India operate a mid-day
meal initiative where nutritious food is cooked at the school and pro-
vided to the students for lunch. The Berambadi school generated GW
from its hand wash (HW) and kitchen wash (KW) sinks. During this
study, the school had about 187 students and 10 staff. The HW sinks
which were used by the students and staff were located at a slightly
higher elevation compared to the KW sink. The GW treatment system
utilized this difference in elevation for gravity flow. This study was
conducted for a total period of one full academic year, which included
50 days of summer break between April and June and a two week
Navaratri/Dasara break at the beginning of October.

2.2. System description

Fig. 1 shows the block diagram of the stages of treatment for HW and
KW greywater. The HW and KW streams were separated owing to the
difference in their composition. The composition of the HW and KW
greywater is discussed in Section 3.1.

The HW treatment module consisted of sink bucket traps with 2 mm
pore size as a pre-treatment stage before the filtration stages. Fig. 2a
describes a picture of the sink strainer used to separate out large food
particles. Following this, three anaerobic bio-filters i.e., concrete tanks
filled with decreasing particle sizes (coarse gravel (20–40 mm), medium
gravel (4–20 mm) and sand (2–4 mm)), were used in the treatment
train. Locally available gravel was chosen as filling material in these
tanks and the tanks were closed to achieve anaerobic biofilm growth
conditions. The three biofiltration tanks (with their lids open) is shown in
Fig. 2b through c. The volume of each of these filter units, their
porosity and hydraulic residence times are tabulated in Table 1. In the
coarse and medium gravel biofilters, the GW feed pipes were brought to
the bottom of the filtration tanks to achieve an upward flow during
operation and to keep these filters partially flooded to facilitate biofilm
growth. The system was not inoculated with any bacteria and was left to
naturally acclimatize. The overflow line from the medium gravel filter
was introduced to the top of the sand filter as shown in Fig. 2c, wherein
the water trickles down through medium gravel biofilter before exiting
from the bottom of the biofiltration tank. The filtered water was then
fed to the aeration tank for aeration.

A recent study had also reported that handwashing soap is the
dominant ingredient in the handwash water [36]. In the handwash area
the students were instructed not to use any soaps as that could poten-
tially increase the nutrient level in the HW water. This practice was not
followed in the school from January to March but was implemented
from July–December.

The KW treatment module consisted of bar screens of 5 mm opening
size and an oil and grease trap as pretreatment stages. The oil and grease
trap, as shown in Fig. 3a, had a detachable perforated (3 mm) basket,
and a secondary chamber where oil and grease can be trapped and
skimmed off. Following this, the KW wastewater was fed to the bottom
of an anaerobic sludge bioreactor (AnSBR) as shown in Fig. 3b. The
overflow from the AnSBR was introduced to the bottom of a biofiltration
chamber. The biofiltration chamber was a stratified column of coarse
gravel (20–40 mm), medium gravel (4–20 mm) and sand (2–4 mm) as
shown in Fig. 1. The coarse gravel was used at the bottom of the biofilter
whereas fine gravel was used at the top. The KW wastewater was made to
flow in upward direction first through the coarse gravel, followed by
medium gravel and finally through sand layers to achieve bio-filtration.
The overflow line from this biofiltration tank was connected to the
aeration tank as shown in Fig. 1.

In the kitchen wash area Vim™ soap was used for utensil cleaning.
The manufacturers claim the composition of Vim soap to be Sodium LAS,
Sodium Carbonate, Neem Oil, Concentrated Lime Juice, CI 74260, CI
11680, and Water [37].

In the aeration tank, the filtered water from HW and KW was mixed
and aerated using locally made diffuse aeration pipes. As shown in Fig. 3c, the aeration system consisted of updraft diffusers, designed using locally sourced PVC pipelines (1 inch dia) perforated (3 – 4 mm) at equal intervals. The aerated water entered the bottom of the ozonation tank through gravity displacement. Cold plasma powered high throughput ozonators, as shown in Fig. 3d, delivering up to 10 gm/hr of ozone were used to achieve ozonation of the GW. The ozonation tank was also fitted with the updraft diffusers to achieve the proper contact of ozone with the GW. The treated GW from the ozonation tank was pumped to the overhead tank daily at 9:30 am. This pumping operation took between 15–20 min. At the end of the pumping, the ozonation tank was emptied to make room for new water to be ozonated. The aeration was performed daily for 90 min from 10 a.m. to 11:30 a.m. The aerated water was allowed to settle for 30–60 min s, before receiving the fresh load, which started after lunch between 1 and 2 pm from HW sinks and between 2 and 3 pm from KW sinks. As the system was gravity fed, the HW and KW water generated in a day displaced the water present in the biofilters and AnSBR. The entry of a new batch resulted in the overflow of the aerated water from the aeration chamber into the ozonation chamber. Ozonation was performed between 3:30 pm and 4:00 pm daily. This treated GW in the ozonation tank was allowed to stay overnight before being pumped to the overhead tank the next morning.

The timings for the operation of the ozonators and aerators were between 12:30 pm and 2:00 pm, during which time most of the day’s GW is generated and channeled through treatment units.

As a first operation step, the treated GW from the ozonation tank was pumped to the overhead tank daily at 9:30 am. This pumping operation took between 15–20 min. At the end of the pumping, the ozonation tank was emptied to make room for new water to be ozonated. The aeration was performed daily for 90 min from 10 a.m. to 11:30 a.m. The aerated water was allowed to settle for 30–60 min s, before receiving the fresh load, which started after lunch between 1 and 2 pm from HW sinks and between 2 and 3 pm from KW sinks. As the system was gravity fed, the HW and KW water generated in a day displaced the water present in the biofilters and AnSBR. The entry of a new batch resulted in the overflow of the aerated water from the aeration chamber into the ozonation chamber. Ozonation was performed between 3:30 pm and 4:00 pm daily. This treated GW in the ozonation tank was allowed to stay overnight before being pumped to the overhead tank the next morning.

The timings for the operation of the ozonators and aerators were between 12:30 pm and 2:00 pm, during which time most of the day’s GW is generated and channeled through treatment units.

2.3. System operation

On a typical working day, the school opens at 8:30 am and closes at 4:30 pm. The lunch (mid-day meals) was served in the afternoon
optimized after quantifying the flow rates in each stage of the system daily, so as to obtain treated water at the beginning of the day and with the least energy consumption. Despite these optimizations, the end quality of water would vary significantly (within the acceptable limits of reuse) due to the high variations in the input parameters to the GW treatment system. Factors such as school attendance, the seasonal variation in the availability of greens and vegetables, guests coming to the school and cultural events in the village affected the GW quality. These external factors were responsible for the variations in the quality and quantity of GW generated, which is discussed in the results and discussion Section 3.

2.4. Sampling methodology

Sampling ports were installed using valves at the end of each stage as shown in Fig. 1. Water samples were collected at each of these ports in a sterilized sample collection container, fortnightly over a period of one year. These samples were analyzed for standard water quality parameters using APHA protocols [38]. Samples were analyzed for pH, total suspended solids (TSS), total dissolved solids (TDS), nitrates (NO₃⁻), total phosphorus (TP), phosphates (PO₄³⁻), temperature (T), biological oxygen demand (BOD₅), turbidity, chemical oxygen demand (COD), total organic carbon (TOC) and fecal coliform counts (FC). The unit of measurement was NTU for turbidity, MPN/100 mL for FC, and ppm for all other parameters. The post-treatment water quality parameters at each stage were compared with the relevant water sewage discharge standards for recycling and reuse [39, 40]. All the data were statistically analyzed using MS Excel Data Analysis tools for statistical measurements such as two-tailed t-test to verify statistical significance. All data is represented in the form mean ± standard deviation (μ ± σ).

The quantity of water consumed at the KW and HW areas were also measured on a monthly frequency to assess the containment loading rates and evaluate the removal efficiencies, due to the separate treatment of these two GW streams.

2.5. Operation and maintenance

One of the major attractions that the system offers is its ease of operation and maintenance. The system has no major machinery requiring skilled labor for operation. The maintenance of the system is only involves cleaning tanks and filters biannually. The detachable perforated basket in the grease trap is washed on a biweekly basis. At present, the system is operated by the school staff after a training and transition period of one year.

The system, therefore, has the potential to be replicated as well as

<table>
<thead>
<tr>
<th>Treatment Stages</th>
<th>Total Volume (L)</th>
<th>Design flow rate (LPD)</th>
<th>Porosity (%)</th>
<th>Hydraulic Retention time (hours)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Handwash biofilters</td>
<td>Coarse gravel biofilter</td>
<td>1014</td>
<td>750</td>
<td>35</td>
</tr>
<tr>
<td></td>
<td>Medium gravel biofilter</td>
<td>1014</td>
<td>750</td>
<td>33</td>
</tr>
<tr>
<td></td>
<td>Sand biofilter</td>
<td>1014</td>
<td>750</td>
<td>28</td>
</tr>
<tr>
<td></td>
<td>Oil and grease trap</td>
<td>148</td>
<td>750</td>
<td></td>
</tr>
<tr>
<td>Kitchen wash filter</td>
<td>Anaerobic sludge bioreactor</td>
<td>1130</td>
<td>750</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Stratified column biofilter</td>
<td>990</td>
<td>750</td>
<td>35</td>
</tr>
<tr>
<td>Aeration tank</td>
<td>Aeration tank</td>
<td>620</td>
<td>1500</td>
<td>9.9 (1.5 h treatment time)</td>
</tr>
<tr>
<td>Ozonation tank</td>
<td>Ozonation tank</td>
<td>620</td>
<td>1500</td>
<td>9.9 (0.5 h treatment time)</td>
</tr>
</tbody>
</table>

As shown in Table 1, the hydraulic retention time (HRT) for the HW and KW treatment modules were 31.2 h and 52 h respectively. The HRT for the overall treatment (including the aeration and ozonation) residence time for treating the HW and KW streams were 51 h (~ 2 days) and 71 h (~ 3 days) respectively.

Fig. 3. Pictures of KW treatment a) Grease trap b) Anaerobic upwelling sludge bioreactor, c) Diffuse aeration system of aeration and ozonation tank d) cold plasma ozonator.
scaled up. Such decentralized plants can also be conveniently built-in urban settings such as apartments, hospitals, and educational institutions, depending on local climatic conditions, population density and land availability.

2.6. Calculations

The removal efficiencies (RE) for the parameters of turbidity, TSS, BOD₅, COD, NO₃, TP, TOC and FC were evaluated for each of the water treatment steps. The removal efficiency of the overall treatment was measured using the percentage reduction in the concentration from the samples collected pre and post-treatment, which is represented by Eq. 1 [41].

Removal Efficiency (RE) = \( \frac{C_i - C_o}{C_i} \) \times 100

(1)

where \( C_i \) and \( C_o \) are the concentrations of the parameters at the influent and effluent samples respectively.

As the aeration and ozonation stages had two inlets with variable flow rates, removal efficiency calculations were measured by load, and not concentration. Load (\( L_p \)) of parameter \( p \) was measured using \( L_p = C_P \times V \), where \( C_p \) and \( V \) is the concentration of parameter \( p \) and volume of the GW.

Removal efficiency of the aeration and ozonation stages for parameter \( p \) was measured using the equation:

\[
RE_p = \left( \frac{Load_{HW} + Load_{KW}}{Load_{HW} + Load_{KW}} \right) - \left( \frac{Load_{out}}{Load_{out}} \right) \times 100
\]

\[
= 100 \times \left( \frac{C_{p,HW} \times V_{HW} + C_{p,KW} \times V_{KW}}{C_{p,HW} \times V_{HW} + C_{p,KW} \times V_{KW}} \right) - \left( \frac{C_{p,HW} \times V_{HW} + C_{p,KW} \times V_{KW}}{C_{p,HW} \times V_{HW} + C_{p,KW} \times V_{KW}} \right) \times V_{out}
\]

(2)

Where \( C_{p,HW} \), \( C_{p,KW} \) and \( C_{p,out} \) are the concentration of parameter \( p \) at the handwash filter outlet, kitchen wash filter outlet and ozonation outlet respectively, \( V_{HW}, V_{KW}, \) and \( V_{out} \) are the volume of GW at the handwash filter outlet, kitchen wash filter outlet and ozonation outlet respectively.

The organic loading rates (OLR) were calculated using the following equation:

\[
OLR = C_{COD} \times \frac{Q_m}{V_p} = \frac{C_{COD}}{HRT_p}
\]

(3)

Where OLR is the organic loading rate in g COD/(m³·day), \( C_{COD} \) is the COD concentration in the input (g COD/m³), \( Q_m \) is the volumetric flow rate of the wastewater (m³/day), and \( V_p \) is the volume of the treatment component. The ratio of the volume of the treatment component and the inflow rate is equal to the hydraulic retention time (HRT).

3. Results and discussions

3.1. Baseline study results

The baseline water quality was measured at the inlet of the school and also at the outlet of the kitchen sink and handwash sink. The baseline study for the GW was done by sampling the water coming out of the kitchen wash (KW) and handwash (HW) areas before mixing, at three different times daily for four days. The average values of the physicochemical and biological parameters obtained in this study are presented in Table 2 along with previously reported values for GW treatment and reuse systems from across the globe.

Compared to the quality of inlet water being supplied to the school (Turbidity = 0.17, TSS = 13, COD = 23 and NO₃ = 34) the water at the GW outlet showed much higher values of Turbidity, TSS, COD, and FC as expected as shown in Table 2.

When the baseline greywater characteristics obtained in this study are compared with the previously reported values, the TSS, BOD₅ and COD values fall within the range of the reported values. These values are higher than most values reported but not as high as the values reported by Mandal et al. which are due to the very low per capita water consumption in Jordan [26]. Halalseh et al. had reported that the high values obtained in their study was due to the high usage of green and vegetables, guests coming to the school and cultural events in the village.

As can be seen from Table 2, there were variations in the quality of the wastewater generated in the school, depicted by the standard deviation in mean values. These variations were due to several day-to-day variations in school attendance and other factors such as the seasonal variation in the availability of greens and vegetables, guests coming to the school, and cultural events in the village.

The water consumption of the school was monitored three times daily for three months as part of the baseline study at three different points. The first point was at the inlet of the school measuring the overall water consumption of the school. The subsequent two points of

Table 2

Baseline greywater characteristics reported in the literature and the data at handwash (HW) and kitchen wash (KW) outlets obtained in this study.

<table>
<thead>
<tr>
<th>S.No</th>
<th>Reference</th>
<th>Location</th>
<th>Grewwater Characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Turbidity (NTU)</td>
</tr>
<tr>
<td>1</td>
<td>[21]</td>
<td>Egypt</td>
<td>105</td>
</tr>
<tr>
<td>2</td>
<td>[22]</td>
<td>Saudi Arabia</td>
<td>103</td>
</tr>
<tr>
<td>3 HW</td>
<td>[23]</td>
<td>Greece</td>
<td>61</td>
</tr>
<tr>
<td>4 KW</td>
<td>[23]</td>
<td>Greece</td>
<td>299</td>
</tr>
<tr>
<td>5</td>
<td>[24]</td>
<td>Brazil</td>
<td>40.4</td>
</tr>
<tr>
<td>6</td>
<td>[25]</td>
<td>Costa Rica</td>
<td>96</td>
</tr>
<tr>
<td>7</td>
<td>[26]</td>
<td>Jordan</td>
<td>845</td>
</tr>
<tr>
<td>8</td>
<td>[27]</td>
<td>Estonia</td>
<td>158</td>
</tr>
<tr>
<td>9</td>
<td>[28]</td>
<td>Uganda</td>
<td>2828</td>
</tr>
<tr>
<td>10 HW sinks</td>
<td>[29]</td>
<td>Brazil</td>
<td>35.8</td>
</tr>
<tr>
<td>11</td>
<td>[30]</td>
<td>India</td>
<td>29.7</td>
</tr>
<tr>
<td>12</td>
<td>This Study</td>
<td>India</td>
<td>196 ± 112</td>
</tr>
<tr>
<td>13</td>
<td>This Study</td>
<td>India</td>
<td>225 ± 118</td>
</tr>
</tbody>
</table>
measurements were before the HW and KW areas measuring the respective consumption in each of these areas. This data was used to calculate the loading factor and reduction of freshwater consumption which is discussed in Section 3.6.

3.2. Pretreatment of HW and KW wastewater using coarse strainers and grease trap

Prior to the slow sand bio-filtration stages, both the KW and HW were directed through separate pretreatment stages. The pretreatment stage was used to alleviate the stress caused by large food particles on the downstream treatment units. Baseline studies clearly indicated the need for installing traps to remove large chunks of food particles which would otherwise potentially clog up the downstream equipment. Also, the KW sink generated GW having high levels of oil and grease. To address this, pretreatment stages were installed.

The HW pretreatment was achieved using particle trapping sink strainers with 2 mm pore diameter as shown in Fig. 2a. These strainers would screen out the large food particles and help in reducing the TSS and turbidity of the GW. The removal efficiency of this stage for turbidity and TSS was evaluated by comparing the baseline GW and the post pretreatment GW. Removal efficiencies (RE) of 88 % for turbidity and 75 % for TSS, was achieved by the pretreatment strainers as shown in Table 3.

The KW treatment module had a grease trapper for pretreatment. The grease trapper was intended to reduce the TSS and turbidity of the GW by trapping the oil and grease present in the KW wastewater. Table 3 shows the turbidity and TSS of the GW from KW sink before and after pretreatment. The KW pretreatment using grease trapper achieved RE of 65 % and 89 % for turbidity and TSS respectively as shown in Table 3. The TSS levels at the outlets of both the pretreatment stages averaged around 80 ppm.

3.3. Performance of biofilters in treating HW greywater

The HW filtration consisted of three separate anaerobic biofilm slow sand filtration chambers. The average organic loading rates in HW filtration system was calculated using Eq. 3 and was found to be ~ 109 g COD/m² day.

Fig. 4 shows the load and removal efficiency of the HW filtration stages over the operational period (the school was not operational during April-June due to summer vacation). Fig. 4a and b shows that the input NO₃⁻ load was between 12 ± 2 g for most of the months and TP load was between 0.75 ± 0.25 g. The NO₂⁻ removal fluctuated between 60 % and 95 % and averaged around 79 %. The removal efficiency of TP fluctuated between the months, but this did not affect the overall performance as TP was low (~ 1 ppm) in the GW from HW wastewater, thereby not impacting its reuse capacity. From Fig. 4c and d it can be seen that, the BOD₅ and COD load for the system were between 70 ± 10 g and 140 ± 20 g respectively. The BOD₅ and COD removal efficiencies were consistent throughout the operational period at around 92 % and 83 % respectively, despite breaks in the operation due to the summer vacation. This indicates that the system is able to perform even with breaks in feed to the biofiltration units.

Fig. 4c and d illustrate the consistent removal efficiency (RE) of BOD₅ and COD throughout the operational period except in November. The low RE of COD in November can be attributed to the relatively lower input COD load on that particular month. The high removal efficiency for the BOD₅ and COD achieved by the HW filtration stages bring the GW within the treated water reuse norms for BOD₅ and COD, details of which are discussed further in Section 3.6.

Fig. 5a and b, shows the picture of the coarse gravel taken during installation and one month after the commencement of the plant operation respectively. From Fig. 5b, the biofilm layer formation after one month of the commencement of system operation is evident on the filter media. The formation of the biofilm layer has been reported to potentially enhance the removal efficiencies through possible several pathways which include biosorption, biological degradation of soluble organics and also reduces odor and color [42]. The microbial communities thriving on the biofilm are known to be responsible for the breakdown of different nutrients, such as phosphorous and nitrogen-containing compounds, carbonaceous materials as well as the removal of trapped pathogens from the wastewater [43,44]. The high removal efficiency observed in this study can be attributed to the presence of these biofilms. The reduction in the TP values may have been influenced by the students not using handwash soaps after the month of July in the handwash areas.

3.4. Performance of AnSBR and biofilters in treating KW greywater

The average organic loading rates in the KW module after the pretreatment were calculated using Eq. 3 and averaged around 179 g COD/m² day, which was about 64 % higher than the HW greywater. As expected, the average organic loading rates from the kitchen sink were higher than that from the handwash sinks.

Fig. 6 shows the load and removal efficiency of the KW module consisting of AnSBR and filtration stages over the operational period. As shown in Fig. 6a and b, the NO₃⁻ and TP load from KW wastewater was between 12 ± 2 g and 4.5 ± 2 g respectively. Although the NO₃⁻ load in the KW stream, was similar to that of HW stream, the TP load was almost double. This high load of TP is attributed to the oil coming from washing of cooking utensils and the cleaning products used in the kitchen. The RE of NO₃⁻ fluctuated between 80 % and 95 % and averaged to be around 88 %. The RE of TP fluctuated between 40 % and 80 % averaging around 69 % and it is believed that the resuspension of biofilms into the water is the reason for such fluctuations. Fig. 6c and d illustrate that the BOD₅ and COD of the KW stream was higher than the HW stream (Fig. 5c and d). This higher load is again believed to be coming from the oil films present on the utensils and washing products used in the kitchen. As can be seen from Fig. 6c and d, the BOD₅ and COD RE stayed consistent throughout the operational period (except February) at around 80 % and 69 % respectively. The exception in RE of BOD₅ for February can be attributed to the lower BOD loads on that month. This trend is similar to that observed in the HW filtration.

The RE of BOD₅ and COD of the KW module was relatively lower than that of the HW module. The AnSBR of the KW module had an HRT of 33 h, and it was noticed that, even after 12 months of operations, there was very little sludge present in the AnSBR.

Typically in the enhanced biological phosphate removal systems COD uptake and P-release occurs in the anaerobic conditions [45]. The exact mechanism of phosphorus (P) removal is yet to be fully understood in an anaerobic system. Past studies have also reported this observation

| Table 3 Turbidity and TSS removal efficiency of pretreatment stages. |
|-------------------|-------------------|-------------------|-------------------|-------------------|-------------------|
|                   | HW before pre-    | HW after pre-     | Avg. Removal Efficiency (%) | KW before pre-     | KW after pre-     | Avg. Removal Efficiency (%) |
|                   | treatment         | treatment         |                             | treatment         | treatment         |                             |
| Turb (NTU)        | 196 ± 112         | 24 ± 12           | 88                           | 225 ± 118         | 78 ± 55           | 65                           |
| TSS (ppm)         | 351 ± 223         | 88 ± 48           | 75                           | 619 ± 237         | 73 ± 30           | 89                           |
and hypothesized two possible explanations. Wang et al. had reported 50–70% phosphate uptake efficiencies in their study and found a correlation between anaerobic uptake of acetate and phosphates [45]. Keating et al. also observed phosphate removal in the anaerobic digestion of wastewater treatment and hypothesized the removal mechanism to be biological in nature, mediated by the biofilms in the reactor [46].

The formation of biofilm was observed in the HW stages of the treatment but could not be monitored in the KW stages due to the filtration units being below the ground level. The odor in the KW water obtained after filtration indicates the presence of anaerobic microorganisms in the KW filtration stages, and biofilm formation is expected in the KW filter. These indicate that the biofilm or anaerobic microorganism in the KW filtration stage is responsible for the TP reduction similar to what was reported by Keating et al. [46].

The KW filtration stages had a lower HRT than the HW filtration stages. This was designed as the AnSBR was intended to be the main component in removing BOD$_5$ and COD. The low sludge level in the AnSBR coupled with lower overall surface area and HRT in the stratified bio-filtration units are believed to be the reason for relatively lower RE for BOD$_5$ and COD.

### 3.5. Performance of aeration and ozonation modules

The filtered water obtained at the end of the filtration stages of both KW and HW systems looked clear as shown in Fig. 7, but was not free of odor. Furthermore, the FC of these samples at the end of the treatment stages was over 1000 MPN/100 mL which exceeds the Karnataka State Pollution Control Board (KSPCB) treated sewage discharge standards [39, 40]. This standard mentions that it applies to recycling and reuse of treated effluent involving human contact [39]. The water obtained at the end of KW filtration did not meet the KSPCB effluent reuse standard norms for BOD$_5$ and COD, which also needed to be addressed. Aeration and ozonation were performed following filtration to resolve these issues. Ozonation enables the removal of odor, color, micropollutants [47] and enhances the disinfection capabilities offered by the treatment plant. The average organic loading rates for the aeration system was calculated using Eq. 3 and averaged around 77 g COD/m$^3$/day.

Ozone generated from on-site plasma sources was used for the final disinfection stage. Despite the higher economic cost of plasma-based ozonation compared to chlorination, plasma-based ozonation results in fewer health impacts relative to chlorination [48, 49]. Ozone is
known, to be superior to chlorine in destroying viruses and bacteria, with contact time of 10–30 min, has no harmful residues, and prevents the biofilm growth and regrowth of microorganisms in wastewater streams [49]. As the treatment module was located in a remote area, a decentralized approach for the disinfection process using ozonation was preferred over chlorination as it would reduce the risks associated with handling and shipping of chlorine [49]. Furthermore, ozonation is also known to elevate the DO concentration as oxygen is the byproduct of ozone degradation [49]. This reduces the required aeration time for the treatment process to achieve safe DO levels.

The ozonators were tailored specifically for the purpose of WW treatment as part of this work. Rao et al. had reported the design and performance of the same ozonator at lower flow rates and ozone outputs (20 LPM flow rate for 1.2 g h$^{-1}$ ozone production) for decentralized GW applications [50]. A 69% reduction in COD was reported upon 30 min of ozonation [50]. For this study, and based on the results reported by Rao et al., the ozonator design was modified and optimized to operate at the higher flow air rates (100 LPM) and produce higher ozone output (4.5 g h$^{-1}$). The ozonator did not require pure oxygen as a feed gas like most of the commercially available ozonators but worked with ambient air as the feed gas. This was more practical as the supply of oxygen cylinders to a rural area is not economically feasible. Furthermore, oxygen cylinders are a fire and explosion hazard in a school, which requires skilled technical labor for operation and maintenance. To achieve a
decentralized system the ozonator was designed to be easy to operate and maintain, requiring no external materials after post-installation. This ozonator was fed air using a compressor, bypassing the need of compressed oxygen cylinders [50]. Four ozonators were placed in parallel as shown in Fig. 3d and operated only for 30 min. daily to achieve the required effluent sewage and reuse standards for the treated GW.

The cost estimation for the disinfection of wastewater using chlorine, ozone and UV varies based on the volume of water treated daily [51]. The cost of treating 1 kL of water using chlorine, ozone, hypochlorite and UV are in the ranges of 0.02–4 $, 0.18–11.7 $, 0.03–4 $ and 0.02–8 $ respectively [49,51–54].

Fig. 8 shows the load and removal efficiencies of the aeration and ozonation stages for turbidity, TSS, BOD and COD over the operational period. The turbidity removal efficiency was consistently between 70 % and 88 % and averages around 83 %. The removal efficiency of TSS, BOD and COD showed variations but averaged around 80, 58 and 49 % respectively. The removal efficiency of BOD and COD may seem low, but it must be noted that this is an enhancement to the high removal efficiencies achieved already by the previous stages.

The FC present in water collected from different sampling points were measured. Table 4 shows the log lower reduction value (LRV) and log higher reduction value (HRV) at different stages of treatment.

The filtration shows an average log reduction of 3.37 and 2.96 in the HW and KW filters. The difference in the LRV and HRV can be attributed to the difference in the FC concentration in the source water.

The ozonator enhanced the disinfection capabilities by reducing the FC by 2 log to the filter-treated water. The water obtained post-ozonation shows very low coliform values 28 MPN/100 ml) and can address these parameters and bring them well within the sewage discharge and reuse standards. The ozonation and aeration stages do not bring the BOD and COD removal of the HW filtration to permissible limits. The KW filtration does not bring the BOD and COD to permissible limits due to poor performance of the AnSBR, higher OLR and lower HRT in the KW bio-filters. The FC in both the KW and HW streams at the filtration outlet was much higher than the sewage discharge and reuse standards. The ozonation and aeration stages address these parameters and bring them well within the sewage discharge and reuse standards.

It is important to note that the ozonation stage increases the NO₃⁻ in the GW, but this range still falls within the sewage discharge and reuse standards. Rahmadi et al. [42] had reported oxidation of nitrate and ammonia to NO₃⁻ leading to an increase in NO₃⁻ upon oxidation. The increase in the NO₃⁻ observed in this study could be due to the oxidation of the other nitrogen species as the overall TN was not significantly affected by the ozonation stage (Refer to Supplementary Material).

Though there are variations in the NO₃⁻ removal over the different months it is important to note that even the untreated streams of HW and KW had NO₃⁻ which were within the permissible limits. The overall treatment system showed RE of 99 %, 98 %, 66 %, 73 %, 98 %, 96 % and >99.99 % in turbidity, TSS, NO₃⁻, TP, BOD₅, COD and FC.

### Table 4

<table>
<thead>
<tr>
<th>Stage</th>
<th>Log LRV</th>
<th>Log HRV</th>
<th>Avg. Log Reduction</th>
</tr>
</thead>
<tbody>
<tr>
<td>HW Bio-filtration</td>
<td>1.22</td>
<td>5.25</td>
<td>3.37</td>
</tr>
<tr>
<td>KW Bio-filtration</td>
<td>1.47</td>
<td>3.86</td>
<td>2.96</td>
</tr>
<tr>
<td>Ozonation</td>
<td>0.59</td>
<td>4.93</td>
<td>1.97</td>
</tr>
</tbody>
</table>

### 3.6. Overall system performance

The water quality parameters at different stages of treatment are shown in Table 5, alongside the treated sewage discharge standards of the KSPCB. It was observed that the pretreatment stages are effective for the removal of turbidity and TSS without which, the slow sand bio-filters would have clogged leading to the requirement for frequent maintenance and increased associated costs. The RE of BOD₅ in the HW pretreatment was around 50 %, which was only 10 % in the KW pretreatment. This could be attributed to the high amount of food waste that was discharged into the handwash area as shown in Fig. 4. This solid food waste was the main constituent responsible for the BOD₅ of the HW stream. In the KW stream, BOD₅ is attributed to several constituents of not all of which were removed by the pretreatment.

The filtration stages of both KW and HW streams reduce the Turbidity and TSS but do not bring them within the sewage discharge and reuse standards. The BOD₅ and COD removal of the HW filtration achieves permissible limits. The KW filtration does not bring the BOD₅ and COD to permissible limits due to poor performance of the AnSBR, higher OLR and lower HRT in the KW bio-filters. The FC in both the KW and HW streams at the filtration outlet was much higher than the sewage discharge and reuse standards. The ozonation and aeration stages address these parameters and bring them well within the sewage discharge and reuse standards.

In Fig. 8 shows the load and removal efficiencies of the aeration and ozonation for a) turbidity, b) TSS, c) BOD₅ and d) COD over the operational period.
respectively. The relatively low removal of NO\textsubscript{3} when compared to the other components can be attributed to the low NO\textsubscript{3} concentrations in KW and HW streams.

### 3.7. Treated water reuse

An average around 667 L of water was treated daily. All of the treated GW was redirected to the toilets for flushing. This corresponds to an annual water saving of 180 kL assuming 270 working days in a year. Fig. 10 shows the treated water generated from HW and KW facilities, individually. The average water consumption in the toilet blocks adds up to be around 754 L daily. Treated GW was utilized and accounted for 85 % of the total water consumption in the toilet block. It has the scope to be also be used for other non-potable purposes.

### 3.8. Performance of the system compared to other reported systems

The system performance was compared with values reported in the literature from field and laboratory studies conducted elsewhere for GW treatment and reuse, owing to a lack of published reports pertaining to the rural Indian context.

Table 6 provides a summary of the previously reported RE values obtained for different parameters upon GW treatment for reuse using different technologies and compares it with this study.

From Table 6 it can be inferred that the GW treatment and reuse system installed at Berambadi government primary school shows RE which is comparable or better than the reported RE values from earlier studies. This indicates that the system installed is performing better than other existing systems in place in different parts of the world in terms of RE. The high RE obtained can be attributed to the integration of different technologies into the system. The RE values obtained in this study have been consistent for almost one year signifying the robustness of this system.

Hydraulic retention time is known to influence the RE values of any given GW treatment system [55]. Detailed analysis on the impact of HRT on the RE of this system has not been performed. As the greywater

### Table 5

Concentrations of different parameters at different stages of treatment.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Hand wash GW baseline</th>
<th>Hand wash GW after pre-treatment</th>
<th>Hand wash GW after triple filtration</th>
<th>Kitchen wash GW after pre-treatment</th>
<th>Kitchen wash GW after anaerobic tank and updraft filtration</th>
<th>GW post aeration and ozonation (end-use)</th>
<th>KSPCB sewage discharge and reuse Standards [39,40]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Turbidity (NTU)</td>
<td>196 ± 112</td>
<td>24 ± 12</td>
<td>4.32 ± 3.84</td>
<td>225 ± 118</td>
<td>78 ± 55</td>
<td>14 ± 12</td>
<td>0.8 ± 0.4</td>
</tr>
<tr>
<td>TSS (mg/L)</td>
<td>341 ± 223</td>
<td>88 ± 48</td>
<td>30 ± 13.2</td>
<td>619 ± 237</td>
<td>73 ± 30</td>
<td>24 ± 15</td>
<td>9 ± 3.1</td>
</tr>
<tr>
<td>Nitrate (mg/L)</td>
<td>34 ± 6</td>
<td>26 ± 14</td>
<td>8.58 ± 6.5</td>
<td>40 ± 6</td>
<td>28 ± 17</td>
<td>9.9 ± 3.4</td>
<td>12.4 ± 11.1</td>
</tr>
<tr>
<td>BOD\textsubscript{5} (mg/L)</td>
<td>344 ± 273</td>
<td>165 ± 72</td>
<td>13 ± 6</td>
<td>445 ± 165</td>
<td>402 ± 178</td>
<td>31 ± 16</td>
<td>9 ± 5</td>
</tr>
<tr>
<td>COD (mg/L)</td>
<td>633 ± 383</td>
<td>328 ± 137</td>
<td>48 ± 38</td>
<td>533 ± 267</td>
<td>497 ± 225</td>
<td>74.3 ± 23</td>
<td>27 ± 16</td>
</tr>
<tr>
<td>TP (mg/L)</td>
<td>1.03 ± 0.68</td>
<td>0.46 ± 0.31</td>
<td>4.53 ± 2.01</td>
<td>1.40 ± 0.62</td>
<td>0.46 ± 0.25</td>
<td></td>
<td></td>
</tr>
<tr>
<td>FC (MPN/100 mL)</td>
<td>2.35 × 10^8</td>
<td>7.2 × 10^6</td>
<td>3.1 × 10^3</td>
<td>2.26 × 10^8</td>
<td>2.8 × 10^6</td>
<td>3 × 10^3</td>
<td>28</td>
</tr>
</tbody>
</table>

Fig. 9 shows the loads and removal efficiency of the overall treatment system over the operational period. From Fig. 10c and d, it can be seen that the RE of BOD\textsubscript{5} and COD is consistent across filtration stages before averaging around 98 % and 96 % respectively. The RE of NO\textsubscript{3} varies between 35 and 80 % and averages around 66 %.

The RE of TP was variable but averaged approximately 73 %.

Fig. 9. Load and removal efficiencies of the overall treatment for a) nitrate, b) TP, c) BOD\textsubscript{5} and d) COD over the operational period.
characteristics at the input described in Table 2 was at the higher range, the effective treatment required a multistage process involving pre-treatment, settling cum filtration, followed by aeration and ozonation. If the GW did not contain high FC values, then there would be no need for ozonation. If the GW was devoid of the KW stream, the implementation of grease trap would not have been required. As this was a rural scenario, space was not a major constraint and a gravity-driven flow was achievable for this system. This may not be possible in the urban context as space constraints may force the system to be underground. There is no fixed system that is optimal for all GW treatment and reuse scenarios. The characteristics of GW to be treated and the location influence the design of the treatment system, as do the needs and capacities of end users and operators.

4. Conclusion

This study reports the performance of a decentralized greywater treatment and reuse system which was operated for over 12 months in a government-managed school in rural India. A greywater treatment train including slow sand biofilters, anaerobic sludge bioreactors, aerators and ozonation system was installed and the performance of each of the subsystems was captured. The results show that

- The pre-treatment reduced the TSS and turbidity effectively thereby reducing the clogging and maintenance in the filtration stages.
- The filtration stages reduced the TSS, turbidity, BOD$_5$, and COD effectively.
- The high FC values at the end of the filtration stages was resolved at the ozonation stages.
- The treated GW obtained after all these stages were well within the range of the effluent discharge standards for reuse with human contact prescribed by the KSPCB.
- The overall treatment system showed RE of 99%, 98%, 66%, 73%, 98%, 96% and >99.99% in turbidity, TSS, NO$_3$, TP, BOD$_5$, COD and FC respectively.

These RE values obtained are comparable and slightly higher than the previously reported values. The decentralized approach using components that require low-maintenance and are simple to operate enabled the system to run smoothly without replacement of system components. The consistent RE for all the parameters discussed for a year of operation signifies the robustness of the system. A total of 180 kL...
of water was saved over the operational period of one-year which was utilized for toilet flushing. This study establishes that a decentralized greywater treatment can be installed and operated with relative ease in a rural Indian setting. The removal efficiencies of each of the sub-systems are quantified which further enables proper selection of these sub-systems based on influent and effluent quality and demand.

CRediT authorship contribution statement

P.S. Ganesh Subramanian: Data curation, Formal analysis, Writing - original draft, Writing - review & editing. Anjali V. Raj: Data curation, Writing - review & editing. Priyanka Jamwal: Writing - review & editing, Funding acquisition. Stephanie Connelly: Writing - review & editing, Funding acquisition. Jagadeesh Yeluriapati: Writing - review & editing, Funding acquisition. Samia Richards: Writing - review & editing, Funding acquisition. Rowan Ellis: Writing - review & editing, Funding acquisition. Lakshminarayana Rao: Writing - review & editing, Funding acquisition, Supervision.

Declaration of Competing Interest

The authors report no declarations of interest.

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Appendix A. Supplementary data

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References


