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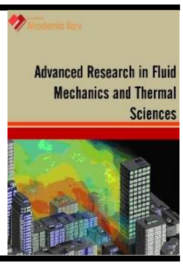

Spatially Constrained Treatment of Raw Sewage to Produce Water for Domestic Use

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ABSTRACT

Water is rapidly becoming one of Pakistan's most precious resources, much more than any hard currency and strength of arms. As the population grows and mismanagement of water sources continues, Pakistan shall move on from a "water-stressed" country to a "water scarce" country. Aiming to resolve just one of the issues with managing water, this project aims to provide an alternative to the average, water-starved inhabitants of Pakistan and provide them with a fool proof method to have a renewable source of clean water to use. Treating sewage is something that is carried out by municipalities all over the world and hence isn't something new. However, there exists no system in Pakistan that allows for households to treat their waste and use it again, and this is the goal of this project. By going over the various steps of primary, secondary and tertiary treatment, aim to bring renewable water sources into people's lives. However, this is not possible using conventional methods and a radical new approach is needed. These methods reduce the time and the space taken to treat water by many factors so that a single unit may fit in a small apartment as well, targeting those who have the least amount of resources to fix their water problem. A prototype consisting of various, state of the art settling tanks, bio reactors and sterilization units was assembled and a continuous treatment process was put together.

1. Introduction

The water is one of the Lord's most precious gift and life around us has evolved around the use of water to sustain itself. The reliance is so profound that all life falters in the absence of water. Human life can survive for days without food, even months, but reaches a critical point within just three days of water starvation. The human bodies are over two-thirds weight percent water, have evolved to use it in every aspect of life. Water can be found everywhere on this planet, in the vast seas, immense oceans, in rivers and lakes both above and below the ground, even in the sky [1,2].

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Distribution of earth's water is shown in Figure 1 [1]. Our planet has its superior method of cleansing and refining water that may be full of impurities. This is vital since not all water is immediately usable to humans, in fact, the vast majority of water from the seas and oceans needs to be treated prior to any practical use [3,4].

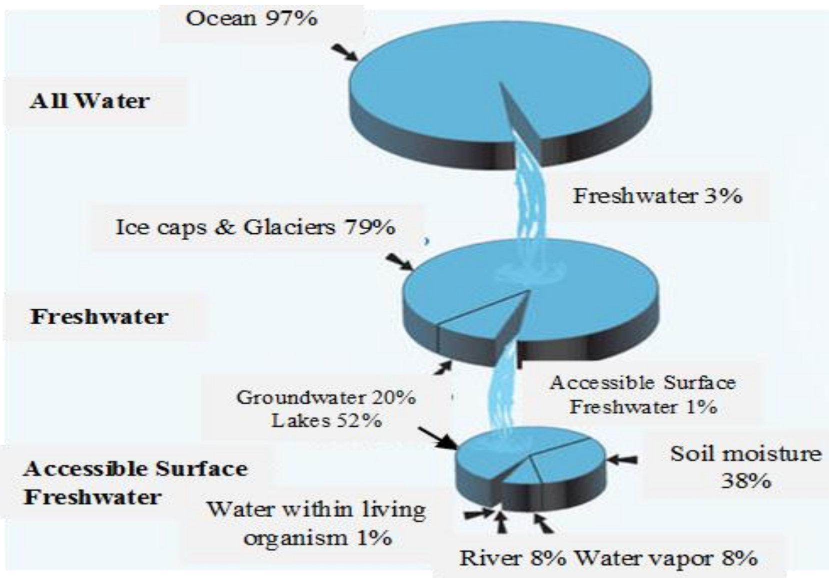


Fig. 1. Distribution of earth's water

This poses a problem because the rate at which humans use up fresh water that they have access to is much greater than the rate at which the planet generates fresh water. Humans are thus resorting to exploring new sources of water all the time, one of the most abundant but rapidly depleting sources is groundwater (water that has sept through the ground and been purified to an extent). With the growing difficulty of finding new water sources, humanity has turned to treat the water it already has access to in order to use it [5]. The extremely large population places a huge demand for fresh water and this demand is only exacerbated by the archaic farming practices that are prevalent in the Pakistan. To make matters worse, the few rivers that do flow in the country are not utilized properly and each year, millions of gallons of fresh water are dumped un-used into the sea [6,7].

The simplification of the problem sought to rectify was to find a way to take raw sewage water, treat it and convert it into the usable water in a domestic, spatially constrained environment. The objective was to provide a solution to the thousands of city dwellers who had very limited access to fresh water but had space and resources to treat their wastewater in usable water and thus regain some of the water that would otherwise be discarded as waste [4,8]. The cost of the project had to be kept low as well since access to clean water is a fundamental human right and charging high prices for clean water is inhumane. High costs would also put the solution out of reach of families with scarcer resources, which is a large portion of the population given Pakistan's median income. Safety was a high concern given the potential use in close proximity of domestic households. Also, since the target was simple households, the operator's skill was assumed to be minimal and thus the project had to be extremely easy to operate with minimal to no moving parts, completely continuous and capable of operating with no supervision [7,9].

The main purpose of this is to protect the water for future generations. When it came to choosing a representative standard, chose the World Wildlife Fund – Pakistan's Water Standards as the baseline and set the targets that wanted to achieve [1,8]. The water can be classified into four classes. Class A public water supply has also categorized into two classes I and II. Class I is the best quality of water. The primary purpose of this is for the water bodies which are uninhibited and have to be protected. This type of water only needs tertiary treatment before it can be consumed for drinking. Class II, this type of water requires a complete process of purification which includes primary treatment, secondary treatment and tertiary treatment to make it fit for drinking purpose. Class B recreational waters are meant for household use and recreational activities like swimming. Class C waters for the propagation of fish and aquatic life and natural water bodies. It is also meant for industrial use and non-human contact uses like in boating. Class D irrigation water is for farms who use water to irrigate their crops for agricultural and livestock uses. It is also used by industries as a medium for cooling [2].

Water standards are based on theoretical and experimental information regarding the pollutants that are present in water and the specified use of water. Water quality is evaluated on the basis of physical, chemical and biological parameters [3,10]. Total suspended solids (TSS) or turbidity is a parameter for the indirect measure of the suspended solids which is based on the light scattering and absorbing properties of the solids. Total dissolved solids (TDS) comprise inorganic salts (principally calcium, magnesium, potassium, sodium, bicarbonates, chlorides, and sulfates) and some small amounts of organic matter that are dissolved in water [11,12]. The pH is a measure of how acidic or basic an aqueous solution is. The pH tells about the presence of hydrogen ion concentration [13].

Biological oxygen demand (BOD) refers to the amount of oxygen that is required to decompose the organic matter in water by the aerobic organisms present in water [12]. Chemical oxygen demand (COD) is a measure of water's capacity to consume oxygen to decompose the organic compounds and oxidation of inorganic chemicals like ammonia and nitrite [14]. Lead gets into the water due to corrosion of service pipe that contains lead. Effects of lead on humans are lowered IQ in children, unhealthy pregnancies and trigger of auto-immune diseases [14]. Total coliforms and fecal coliforms are commonly used as indicator organisms for fecal pollution [15]. Fecal coliforms arise from the gut and waste of animals and humans [1]. After reviewing all the different water standards and keeping in mind the constraints of space, technology and cost it was concluded that sewage treatment would treat the water and bring it to the standard of Class B which is the criteria for recreational water. Waters for this class are intended to be for primary contact recreation such as bathing, swimming and skin diving [8]. The mode of operation, as the unit would be installed inside the house and it would be impossible to continuously oversee the process, it was decided to use a continuous mode of operation instead of a batch operation. It would require minimum supervision [11].

1.1 Conventional Wastewater Treatment Process

Figure 2 represents wastewater treatment, or sewage treatment is the process employed to make water of specified quality by removing contaminants from wastewater, fundamentally from household sewage. Physical, biological and chemical processes are used to remove contaminants and produce water that should comply with the respected standards of either dumping into the large reservoirs like; river, lakes and seas or able to be reused for the specified application. It typically involves three steps with an additional prerequisite step of preliminary treatment the other steps are primary, secondary and tertiary treatment [5].

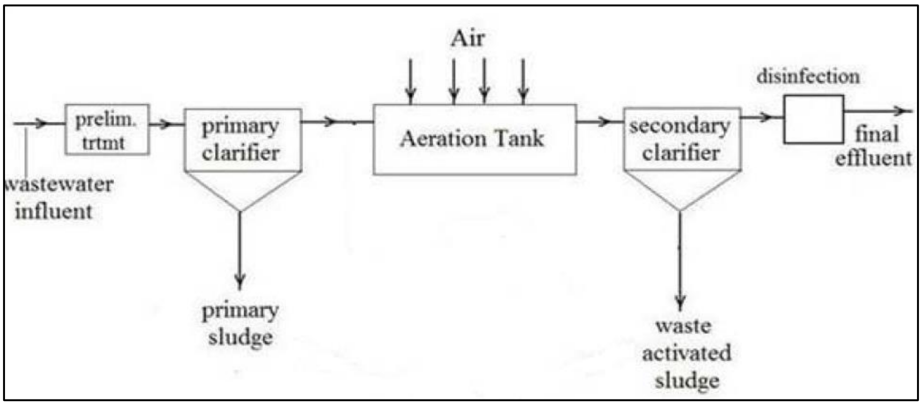


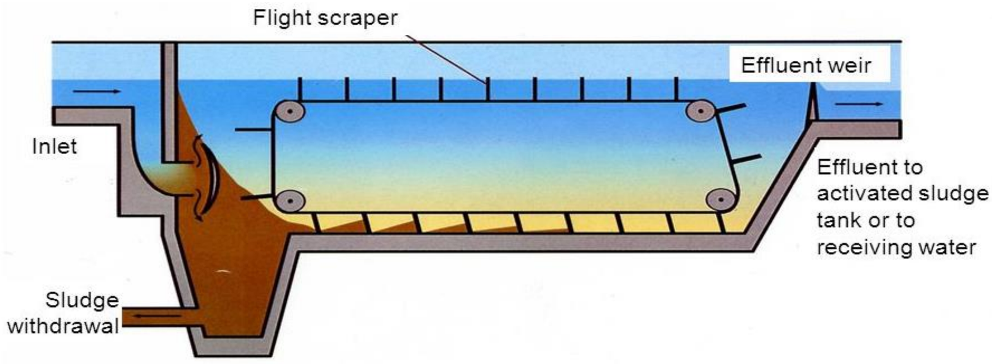
Fig. 2. Wastewater treatment process schematic

It is the pre-requisite step to make water stream suitable to be operated in subsequent treatment processes. In the first place, it involves the screening process. Other special equipment is also employed to remove grits that get washed in the sea water [14]. Primary treatment involves the separation of slits, clay, small size sand particles and solid organic matter (including human excrete) from the wastewater. These settled solids are called as 'Sludge'. There are scrappers at the bottom site to continuously push the solids towards bottom outlet to be removed. These solids than either processed before dumping or dumped directly at the disposal sites. The rest of the treated water overflows and moves to the downstream secondary treatment [5,14]. Secondary sludge is formed in this process that is needed to be removed before sending this water to the tertiary treatment. So, another secondary clarifier is needed at its discharge that settles the sludge and remove it [11,14]. Tertiary treatment includes any treatment other than primary or secondary that makes effluent water qualify for its end purpose limit. It typically involves disinfection of water, either, by chemical addition or by UV-rays. It may also involve other processes to meet stringent water standards like nitrification for nitrogen reduction or any other poisonous metals like arsenic and mercury [5].

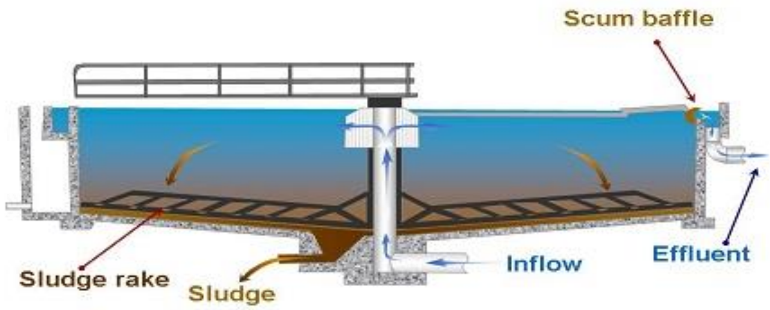
In rectangular settling tank, a long horizontal quiescent passage for liquid flow is provided. The flocculated water stream is entered at one side, flows through the tank, where particles fall with their falling terminal velocities along the path and finally leaves from the other side. Belt type moving Scrappers mounted at the bottom that removes the falling solid sludge continuously. The baffle plate is provided at the inlet to mitigate the turbulence effect of the inflow stream [16].

Figure 3(a) shows long rectangular basins are hydraulically more stable, and flow control for large volumes is easier with this configuration, however, to make a quiescent environment they need to have large cross-sectional area or volume [17]. A long rectangular settling tank can be divided into four different functional zones. Circular settling tank has the same functional zones as the long rectangular tank, but the flow regime is different as shown in Figure 3(b). In the circular tank, the flow enters at the center and is baffled to flow radially towards the perimeter, the horizontal velocity of the water is continuously decreasing as the distance from the center increases. However, the flow arrangement is not very simple and difficult to achieve. They are suitable only at the large capacity treatment units [17,18].

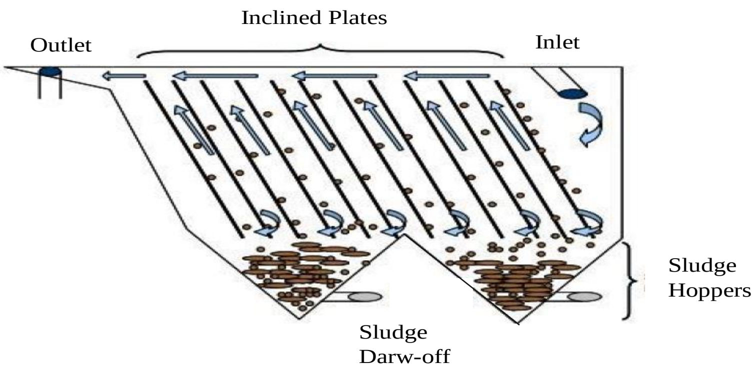
Figure 3(c) displays lamella settler or inclined plate settling tank, often used at the place of conventional sedimentation tank. The settled solids on the plates slide down due to the inclination provided and collected at the bottom hopper sludge removal section [17]. The main advantage of lamella clarifiers over other clarifying systems is the large effective settling area caused by the use of inclined plates, which improves the operating conditions of the clarifiers in a number of ways as given in Figure 4. Operation within an enclosed space also allows for better control of operating temperature and pressure conditions. The inclined plates mean the clarifier can operate with overflow rates 2 to 4 times that of traditional clarifiers which allow a greater influent flow rate and thus a more time efficient clarification process [13,19].



(a)



(b)



(c)

Fig. 3. Settling tank (a) Rectangular (b) Circular and (c) Lamella settling tank

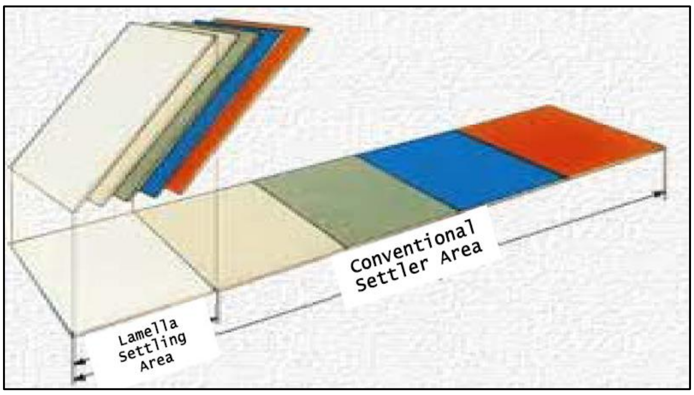


Fig. 4. Settling area enhancement

The governing parameters for the selection of settling tank based on the project constraints are given in Table 1.

Table 1
Analysis of various settlers on prescribed criteria's

Parameter	Circular Tank	Rectangular Tank	Lamella Clarifier
Foot print	✕	✕	✓
Operational cost	✓	✓	✓
Flow controllability	✕	✓	✕
Coagulation	▼	▼	✓

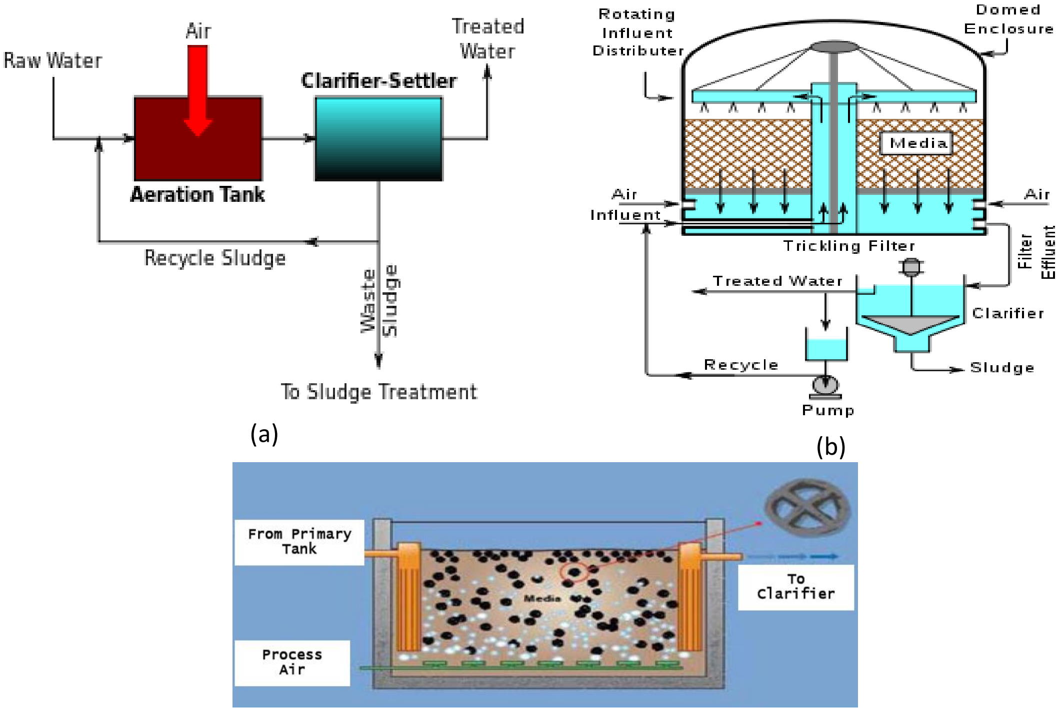
Foot print, utilizing multiple trays in a lamella clarifier makes it effective in terms of the space requirement over its other counterparts and makes itself the most appropriate for the given application. The benefits obtained as faster settling by using coagulant chemical provides a decent trade-off and still justifies Lamella selection over its other counterparts [13,19].

Activated sludge is a suspended growth secondary treatment process that primarily removes dissolved organic solids as well as settle-able and non-settle-able suspended solids. The settled solids and microorganisms are pumped back to the front of the aeration basin, while the clarified water flows on to the next component [20,21]. Figure 5(a) portraits a simple schematic of an activated sludge system. Figure 5(b) represents the trickling filters (TF's), are used to remove organic matter from wastewater. The TF is an aerobic treatment system that utilizes microorganisms attached to a medium to remove organic matter from wastewater [22]. The Moving Bed Biofilm Reactor (MBBR) is another attached growth biological process as a portrait in Figure 5(c). That is, the microorganisms that carry out the process are attached to a solid medium, as in the trickling filter system but by contrast, in a suspended growth biological waste water treatment process, like the activated sludge system, the microorganisms that carry out the treatment are kept suspended in the mixed liquor in the aeration tank [20,23]. Activated sludge system is comparatively more efficient than trickling filter while MBBR is the most efficient among them all because of no channeling of fluid and also the air to water exposure is maximum which helps in the better generation of bacteria [24].

Chlorination of water is the most common and widely used method used since the 19th century. Chlorine is very effective in destroying pathogen, bacteria, protozoa and viruses. Chlorine is used as a gas or as chlorinated compound like sodium hypochlorite. When dissolved in water, chlorine converts to hydrochloric acid and hypochlorous acid [25].

$$Cl_2 + H_2O \rightleftharpoons HOCl + HCl \tag{1}$$

This process is cheap and easily available, control odor and septicity destroy cyanides and phenols and disinfection of salmonella and cholera. Although chlorination is cheap and easy, it comes with a lot of drawbacks. During chlorination the by-products such as tri-halomethanes and halo acetic acids which are responsible for health hazards. If the exposure time is a lot and these byproducts are consumed in high amount, then it affects the brain activity. The by-products are carcinogenic and contribute to kidney and liver cancer [5].



(c)

Fig. 5. Water waste treatment system (a) Activated sludge system (b) Trickling filter system and (c) Moving bed bio reactor basic design

In ultraviolet treatment, UV rays penetrate the cell walls of the bacteria and destroy the DNA and RNA of the cell. Due to which the bacteria is not able to reproduce. The parameters that contribute to the effectiveness of UV treatment are the time duration of exposure to waste water, radiation intensity of UV, waste water characteristics and specifications of the UV reactor. Mercury lamps are used which emits UV radiation of range 250 nm – 270 nm. This wavelength possesses germicidal properties to kill bacteria. UV treatment can be direct or non-direct contact with waste water. The advantages of UV treat ment are that it is very potent against most of the viruses, cyst and spores. Unlike chlorination which is a chemical process, UV is a physical process which does not add any harmful chemicals to the water for disinfection. Hence it is much safer and easier to handle. The equipment of UV treatment takes up less space and the time of contact is 20-30 seconds which is very less as compared to other disinfectants. One major drawback of UV treatment is that due to the high concentration of suspended solids present in water it becomes ineffective. So, the water has to be treated for removal of suspended solids. Another drawback is that in places where UV does not reach, bacteria can start to grow [12].

The method of producing ozone involves providing enough energy to oxygen molecules that they dissociate into separate molecules and when they collide with other oxygen molecules, forms an unstable gas called ozone O_3. Usually, ozone is produced near the wastewater treatment plant. Ozone exhibits strong germicidal properties. It works by attacking the cells of bacteria. Disrupts the nucleic mechanism of the cells and disrupts the cell wall [10]. In comparison to chlorination, ozone possesses greater destroying properties of killing bacteria. Also, the contact time of only 10-30 minutes is required. Ozone does not add any harmful by-products to water which might cause harm to humans. After the water is disinfected with water, there are very fewer chances that bacteria might regrow in water. The drawbacks of ozone include; not proper inactivation of all viruses and bacteria due to less exposure time. Ozone possesses corrosive properties so the piping system and tanks might be damaged. Ozone also poses a safety risk to the works on site as it is irritating and toxic to humans. The comparison between different tertiary techniques is given in Table 2.

Table 2
Analysis of comparison between different tertiary techniques

Analysis	Chlorination	UV Treatment	Ozone
Effectiveness	Medium	High	Medium
Safety	Low	High	Medium
Exposure time	High	Low	Medium
Harmful by-products	High	-	-

2. Methodology

The initial 'dirtiness' of the water needed to be judged and hence the amounts of compounds both organic and inorganic need to be factored.

2.1 Parameter Set Testing

According to the water standards that were chosen there are two physical parameters, sixteen chemical parameters and two biological parameters. Sewage water samples were taken from a representative site at multiple times of the day and samples sent to the Pakistan Council of Scientific and Industrial Research, Karachi (PCSIR) for testing of those parameters which could not be tested in labs within NED University of Engineering and Technology. Parameters tested within the NED University of Engineering and Technology, results are shown in Table 3.

Table 3
Results of tests conducted within NEDUET

Parameter	Unit	Proposed value	Tested Average
	Physical parameters		
Total dissolved solids (TDS)	mg/L	1000	800
Electrical conductivity at 25°C	S/m	1.50	0.89
	Chemical parameters		
pH	-	6.5-8.5	6.9
	Biological parameters		
Coliform bacteria (fecal)	No./100 mL	200	1000
Coliform bacteria (total)	No./100 mL	1000	15,000

Results from independent testing sources are shown in Table 4.

Table 4
Results of tests conducted Pakistan Council of Scientific and Industrial Research, Karachi (PCSIR)

Parameter	Unit	Proposed value	Tested Average
	Chemical parameters		
BOD	mg/L	8	220
Ammonia	mg/L	1.0	1.7
Phenolic compounds as phenol	mg/L	0.01	~
Sulphates	mg/L	400	150

Some values from the tables are over the accepted limit and hence represented the aspects that needed to focus on. Unsurprisingly, the raw bacteria count from untreated sewage was astronomical compared to what was needed, thus highlighting a need for a very efficient method of reducing bacteria count. The next thing that stood out was the BOD of raw sewage was around 2750% greater than it should be for it to be considered as treated. This meant significant BOD reduction was needed and something highly efficient was in order.

The dissolved oxygen count was higher, but that was not of major concern since the samples collected had been open to oxygen for an unspecified time and had micro-organisms in it that produced oxygen. They would not be of concern once treated for BOD reduction. The ammonia content represents the amount of nitrates that were dissolved in the sample and tests showed that the values were only 170% more than expected values, so this process would be able to handle that reduction. The values that stood out truly were the TDS or Total Dissolved Solids. Water from ground water sources usually have high TDS values and expected something similar for the tests due to the mixing of ground water during household use, but the value of 800 mg/L was below the threshold, to begin with, and steered this project away from employing extra reduction methods. The lack of TDS was displayed when the sample showed negligible amounts of sulphates, arsenic and heavy metals.

2.2 Determining the Settling Rate

While it was impossible to test all parameters during this project, the calculations are based on had to rely the values obtained from literature. The use of a flocculant was one of those parameters. Decisions on what flocculant to use and how much was made based on results obtained from a literature and are shown in Figure 6.

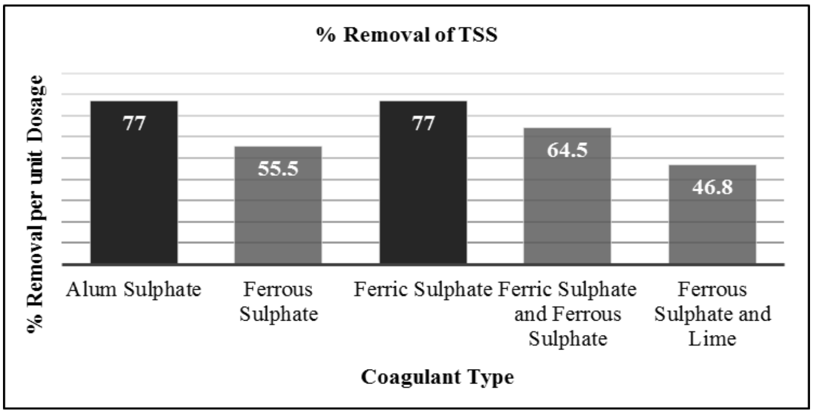


Fig. 6. Suspended solid removal via different coagulants

Based on this the two best options were identified as alum sulphate and ferric sulphate, both of which achieved similar removal of suspended solids for the same amount of dosage is shown in Figure 7. So, the concentration of the amount of flocculant to be added turned out to be the most optimum at around 80 mg/L to 100 mg/L. At values greater than 100mg/L, higher percentage removals were seen, but the increase in removal did not scale up to the increase in flocculant dosage. The settling rate experiments result was verified in the lab by measuring absorptivity of light due to suspended particles in the sample at different concentrations and times and the results concur with literature as presented in Figure 8.

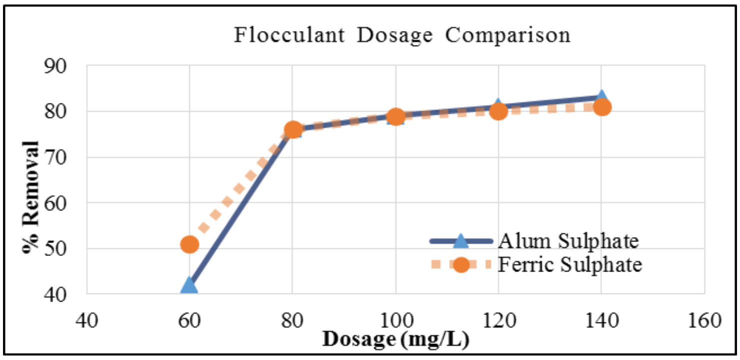


Fig. 7. Comparison of coagulant dosage

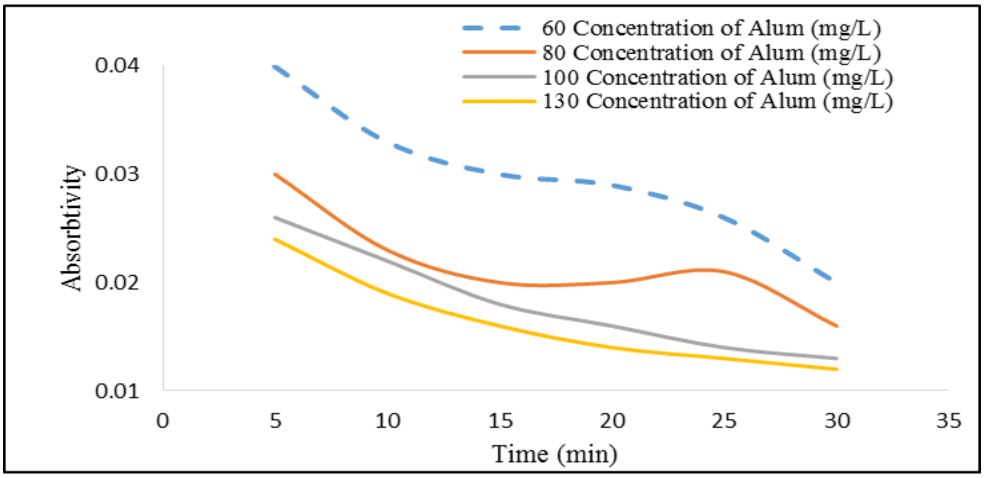


Fig. 8. Spectrophotometer test results plot

Based on the commercial availability in Karachi and a general market survey, the project decided to use aluminium sulphate as the flocculant. The next step was to determine the rate at which the flocs formed and the rate at which they fell. A generalised test which measured the concentration of particles in a graduated cylinder as time progressed gave a good estimate of the time it took for a sample of sewage water to settle as shown in Figure 9.

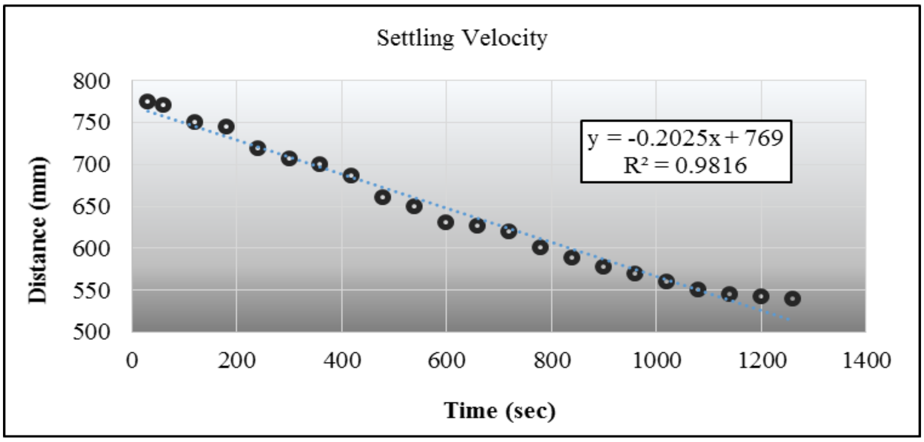


Fig. 9. Plotting to find settling velocity

The results of distance against the time are plotted above which gave an equation "y= -0.2025x+769" where the 769 is the y-intercept and merely represents the height to which the settling cylinder was initially filled. The gradient, -0.2025 has the units of mm/sec and is negative to show the direction of velocity, downwards. Figure 10 shows the samples and effects of different coagulant dosage.

(a) (b)

Fig. 10. Coagulant Dosage (a) Samples being tested with various dosage of coagulant, (b) Effects of different coagulant dosage with time

2.3 Designing the Lamella Clarifier

The designing objectives of Lamella settling tank are to determine the required settling projected area of plates for the given reduction of TSS or NTU and the required volume of settling zone to provide an adequate residence time for the particles to settle down.

Initially "Jar Tests" has been performed for the determination of optimum coagulant (alum) concentration to reduce the TSS expressed in terms of Turbidity under 5 NTU to meet the specified selected water quality standards as shown in Figure 11. The concentration of 100 mg/L has been chosen based on results.

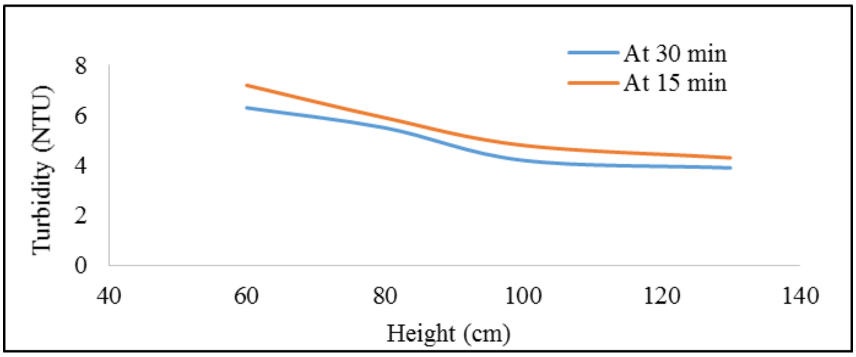


Fig. 11. TSS test in term of turbidity

On the basis of selected alum concentration settling velocity was determine by observing the surface settling depth versus time relation in a vertical cylindrical column. Settling velocity has been found to be "0.8 m/h" or "19 m/d". The necessary condition for the settling to be done up to a given particle removal is given by $SOR < Vt$, where V_t is the terminal falling velocity and *SOR* is the surface area loading rate, it is given by Eq. (1), defining *SOR* formula;

$$SOR = \frac{Vol.Flow\ Rate}{Projected\ Area} = \frac{Q}{A} \tag{2}$$

Based on the literature review, in order to make sure adequate required settling, V_t can be taken roughly as 8 times of SOR to make above equation equality [16]. Therefore; Eq. (2) shows are governing equation equality.

$$Vt = 8 * SOR \tag{3}$$

So, taking V_t as 19 m/day and Q as 160 litres/day in the above equation gives Eq. (4) shows area for projected value.

$$A = 2.4\ ft^2 \tag{4}$$

Taking Lamella plates dimensions as 1.2*1 ft^2 and plate angle of inclination of 60 degrees, based on a literature review for sufficient sliding of sludge from plates. The number of plates required (*N*) has been found by the Eq. (3) [16].

$$N = \frac{projected\ area}{plate\ area * \cos 60} = \frac{2.4}{1.2*0.5} = 4 \tag{5}$$

So, 4 no of lamella plates of dimensions 12*1 ft^2 are required for the settling tank. Based on the literature review, the typical residence time for sufficient settling is in the range of "0.33 h to 13 h". So, settling zone volume has been calculated taking sufficient residence time of "6 h" for designing purpose. Settling zone volume (*V*) is given by Eq. (6) [18].

$$V = (Vol\ Flow\ Rate) * (Residence\ Time) = Q * t \tag{6}$$

Therefore, putting *Q* as 160 litres/day and *t* as 6 hours or 0.25 day, the volume is 40 litres. So, 40 litres settling zone along with adequate additional sludge hopper volume is required for the settling tank.

2.4 Designing the Moving Bed Bio-Reactor

An MBBR process could be worked as a pre-treatment to another secondary treatment process to overcome the overloading of a secondary treatment process or as only a secondary treatment process. The key design parameter for both the cases is same that is surface area loading rate (SALR), with unit g/m^2/day that is loading rate of BOD per m^2 of carrier surface area. By using the selected flow rate for wastewater and the BOD in the influent, can determine the bod loading rate of wastewater in g/m^2/day and then dividing this loading rate by SALR the carrier specific surface area in m^2 can be calculated. The carrier fills %, carrier specific surface area and carrier percentage void space can then be used to calculate the required carrier volume, tank volume and volume of liquid in the reactor [20]. The given Eq. (7) is used in calculations.

$$BOD\ loading\ rate = Q * So * 8.34 * 453.59 \tag{7}$$

where *Q* is the flow rate of waste water in MGD, S_o is the concentration of BOD in influent with unit mg/l, 8.34 is the conversion from mg/l to lb /MGD and 453.59 is the conversion factor from lb to the gram. After collecting samples from different waste water flowing streams and conducted BOD tests from the lab the BOD was found to be 220 mg/l. For the purposes of over designing apparatus and allowing for fluctuations in influent concentrations, a designed reactor based on a higher value [26]. This process scaled down to 160 l/day to keep the tank volume less, although this process has full ability to be run on much greater capacity than the supposed one. Taking influent flow rate as Q=160 g/l and *So* = 250 mg/l gives BOD loading rate=39.974 g/day. The required carrier surface area is calculated by the Eq. (8).

$$Required\ carrier\ surface\ area = \frac{loading\ rate\ of\ BOD}{Surface\ area\ loading\ rate} \tag{8}$$

where BOD loading rate is in g/day, SALR is the design surface area loading rate in g/m^2/day. The selection criteria for SALR (surface area loading rate) and typical design values of SALR for removal of BOD obtained from the literature are given in Table 5.

Table 5
% BOD removal and SALR relations

BOD Removal Rate	% Removal of BOD	Design SALR g/m^2/day
High flow rate	75-80	25
Normal flow rate	85-90	17
Low flow rate	90-95	7.5

The g/day in SALR unit shows the parameter being removed and the m^2 in SALR unit shows the surface area of the carrier required. As BOD is 8 mg /l and influent BOD is 250 mg/l, SALR selected for this % removal is 7 g/m^2/day, using BOD loading rate calculated. The required carrier surface area=5.714 m^2. Carrier's specific area is quite large and required a fair bit of design and consideration [21].

$$Required\ volume\ of\ carrier = \frac{required\ carrier\ surface\ area}{carrier\ specific\ surface\ area} \tag{9}$$

where required carrier surface area in m^2, carrier specific surface area is in m^2/m^3. Using the values, calculated required carrier surface area =12.5 m^2 and the required volume of carrier = 0.017 m^3 from Eq. (9) and (10).

$$Required\ tank\ volume = \frac{required\ volume\ of\ carrier}{carrier\ fill\ \%} \tag{10}$$

where tank volume is in m^3. Using the required volume for carrier = 0.03 calculated in the previous step, the carrier fill percent = 30%. As these chips locally are designed, the best carrier fill % get was 30 %. The required tank volume = 0.0554 m^3.

Void fraction is a fraction of volume of voids by total volume, or it is the percentage of void spaces in the material. So, the hollow chips for this case, the formula for the void fraction becomes as written in Eq. (11) [20].

$$Void\ fraction = \frac{inner\ volume}{outer\ volume} \tag{11}$$

where inner diameter of chips = 2.1 cm, outer diameter of chips = 2.7 cm, inner volume = 1.73 cm^3 and outer volume = 2.8 cm^3. The void fraction is 0.619.

$$\begin{aligned} &Liquid\ Volume\ in\ Tank \\ &\quad = required\ tank\ volume - [required\ carrier\ volume * (1 - void\ fraction)] \end{aligned} \tag{12}$$

where carrier volume = 0.017 m^3, tank volume = 0.0554 m^3 and void space = 61.9%. The required liquid volume in tank = 0.0491 m^3 or 49.1 litre obtained from Eq. (12). MBBR retention time based on the principles of chemical reaction engineering, the time needed to process this much amount of liquid is given by Eq. (13) [24].

$$Retention\ Time = \frac{liquid\ volume\ in\ tank}{inlet\ flow\ of\ waste\ water} \quad (13)$$

where liquid volume in tank = 49.1 ltr, waste water influent= 160 litre/day, which gives the retention time = 7 hrs and 22 minutes. The primary requirement of an aeration system is the air introduction through an air compressing device. The pressure calculation for the air compressor is as below in Eq. (14).

$$Guage\ Pressure = (Specific\ weight) * (height) \quad (14)$$

where specific weight = 62.4 lb_f/ft^3, height is 1.2 ft. The Eq. (15) for gauge pressure computation

$$Gauge\ Pressure = \left(62.4\frac{lbf}{ft3}\right) * (1.2\ ft) * \left(1\frac{ft2}{144in2}\right) = 0.52\ psig \quad (15)$$

The Eq. (16) for total pressure formula

$$Total\ Pressure = Gauge + Atmospheric \quad (16)$$

The total pressure is 15.22 psi. According to the literature review, higher pressure than calculated pressure compressor should be employed to overcome frictional losses [23,24]. Computing the required air flow rate for aeration for the given application is a cumber stone task. It requires both the experimentation and theoretical background. Initially, the experimentation has been done to check the appropriate chips suspension and flow circulation. On the basis of these experimentation result, calculations have been done by taking the experimental result as a basis for an initial value for an iterative optimum flow determination method. Initially experimentally determined value has been found to be 50 GPM. At this air flow rate, sufficient flow circulation and chips suspension have been observed. So, it is taken as the initial value for iterative calculation. The calculation for hierarchy, taking the initial flow rate Q_a then find superficial velocity by Eq. (17).

$$U_a = \frac{Q_a}{A} \quad (17)$$

where A is the flow area, given by Eq. (18).

$$Area = \frac{\pi N d^2}{4} \quad (18)$$

Then mass transfer coefficient by Eq. (19) [14].

$$K_a = 0.269 x {U_a}^{0.082} \quad (19)$$

Then standard oxygen rate by Eq. (20) [21].

$$SOTR = K_a(DO_{sat} - DO)V \quad (20)$$

where V is the volume of water in Tank. Then the air mass flow rate is calculated by Eq. (21) [18].

$$Air\ Mass\ Flow\ Rate = \frac{SOTR}{0.233} \quad (oxygen\ is\ 23\%\ by\ mass\ in\ air) \quad (21)$$

Then find the air volumetric flow rate by Eq. (22) [14].

$$Volumetric\ Flow\ Rate = Q = \frac{mRT}{MP} \tag{22}$$

Convergence will occur if the successive iterative values start giving almost the same value. Table 6 shows the list of variables used to calculate the flowrate and Table 7 gives the iterative results.

Table 6
List of variables used to calculate the flowrate

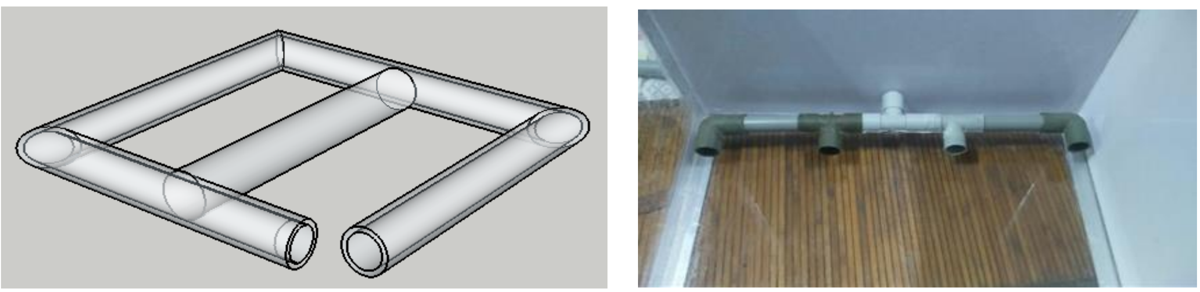
Symbol	Value Used	Symbol	Value Used
R	8.314 J/mol.K	DO	1.4 mg/L
T	298 K	DO_{sat}	8.4 mg/L
P	101325 Pa	Mw.air	29 g/mol air
V	55 liters	Calculated Area	0.0000565 m^2

Table 7
Iterative results of flowrate calculation

Qa GPM	Ug m/s	Ka sec^{-1}	SOTR mg/sec	m mg/sec
50	52	0.37	143	614
35	36	0.32	140	603
33	33	0.31	138	600
32.7	32.2	0.31	138	600

Therefore, approximately 33 GPM flow rate is required, to include the discrepancy between theoretical and practical aspects, an over specified flow rate of 40 GPM has been selected.

Figure 12 presents a basic design of the manifold was modelled that would fit in the base of the Moving Bed Bioreactor. The flow rate of the air that needs to pass through it has been calculated as well as the total area that was needed to deliver the air. Area = 0.0000565 m^2 = 56 mm^2. The manifold is designed for orifice size of diameter 2 mm, giving each orifice an area of 3.14159 mm^2. Thus, the number of orifices is given by: $\frac{56}{3.14159} \approx 17\ orifices$. These holes were drilled into the manifold and set up at the base of the MBBR. A lot of thought was given to the chip design. There are almost no commercially available variants of Bio Reactor chips available in Pakistan. Figure 13 shows of commercially available chips. Potential sources all pointed to suppliers existing in neighbouring China, but this seemed like a very expensive and time-consuming process. It also went against the goal of making a completely homogenous project.



(a) (b)

(c)

Fig. 12. Designing Manifold (a) 3D model of Manifold, (b) Constructing the manifold, and (c) Manifold constructed and set up

Fig. 13. Commercially available chips

Chips are specified and sold according to a characteristic that is called the Chip Specific Area. This is a simply a way of measuring the surface area to volume ratio of an object and is given by Eq. (23) [20].

$$Specific\ Area = \frac{Surface\ Area\ of\ Object}{Volume\ of\ Object} \tag{23}$$

In order to design the chips, find a way to match the specific area of commercially available chips. Market surveys online showed that Chinese chips had on average a specific area of around 420 to 500 m^{-1}. So, designed a chip simple and easy. The process started with a basic circular chip and noticed that it had a rather low specific area. However, further analysis showed that the surface area changed a lot slower compared to the volume as the size of the chip changed if the thickness of the chip was kept constant. This leads to doing a few calculations based on some very easily commercially available tubes and arrived at some numbers (Tube outer diameter=1 inch=26 mm, Tube inner diameter=21 mm and Tube length=1 inch=26 mm).

$$Inner\ Surface\ Area = 2\pi\frac{ID}{2}l = 17.15\ cm^2 \tag{24}$$

$$Outer\ Surface\ Area = 2\pi\frac{OD}{2}l = 21.81\ cm^2 \tag{25}$$

$$Surface\ Area\ on\ Side = 2\pi\left(\frac{OD-ID}{2}\right)^2 = 4.27\ cm^2 \tag{26}$$

This gives a total surface area of 43.23 cm^2. Volume calculations are given in Eq. (27)

$$Volume = \left(\pi\left(\frac{OD}{2}\right)^2 l\right) - \left(\pi\left(\frac{ID}{2}\right)^2 l\right) = 5.56\ cm^3 \tag{27}$$

Thus, the surface area to volume ratio is

$$Specific\ Area = \frac{17.5+21.81+4.27}{5.56} = 2.968\ cm^{-1} = 296.8\ m^{-1} \tag{28}$$

From this graph, see that as length decreases the specific area shoots up quite high. For chips of length 4 mm, the specific area approaches commercial values. However, making chips this small was not feasible mechanically for us so as a compromise, chips of length 10 mm were chosen to give us a specific area of 343.8 m^{-1}. This is the origin of the value used above in the calculations are summarised in Table 8. Plotting a graph of a specific area to differing chip lengths is shown in Figure 14.

Table 8
MBBR designing calculations

BOD Loading Rate

Water Flow Rate in L/Day	160	L/Day	
Waste Water Flow Rate - In (MGD)	4.23E-05	MGD	
BOD Concentration (mg/L)	250	mg/L	40.0 g/Day BOD Loading Rate
mg/L to lb/MG	8.34		
lb to g	453.59		

Required Surface Area

Initial BOD	250	mg/L	
Target BOD	8	mg/L	5.714 m2 Required Surface Area
% reduction	97	%	
SALR (g/m2/day)	7	(g/m2)/day	

Required Carrier Volume

Carrier Specific Area	343.767	m2/m3	
Surface Area Needed	5.714	m2	0.017 m3 Carrier Volume

Required Tank Volume

Carrier Vol	0.017	m3	
Carrier Fill %	30	%	0.0554 m3 Tank Volume
			55.409 L

Liquid Volume in Tank

Carrier Vol	0.017	m3	
Tank Volume	0.0554	m3	0.0491 m3 Liquid Volume
Void Space	61.861	%	49.069 L

Retention Time

0.307	Day	
441.619	Minutes	7 Hours 22 Minutes
	%	

Circle

	Diameter(cm)	Radius(cm)	Length(ft)	Unit Price(Rs)	Total Price (Rs)
Inner	2.1	1.05	97.334	17.0	1654.68
Outer	2.67	1.335			

			CHIPS		
Inner Volume	3.464	cm3	Inner SA	6.597	cm2
Outer Volume	5.599	cm3	Outer SA	8.388	cm2
			Side SA	4.271	cm2
chip size	10	mm			
no. of chips	2967.50		Total SA	19.256	cm2
chip vol	5.60	cm3			
Void Fraction	0.619		Surface/Volume	3.438	\cm
			Specific Area	343.767	\m

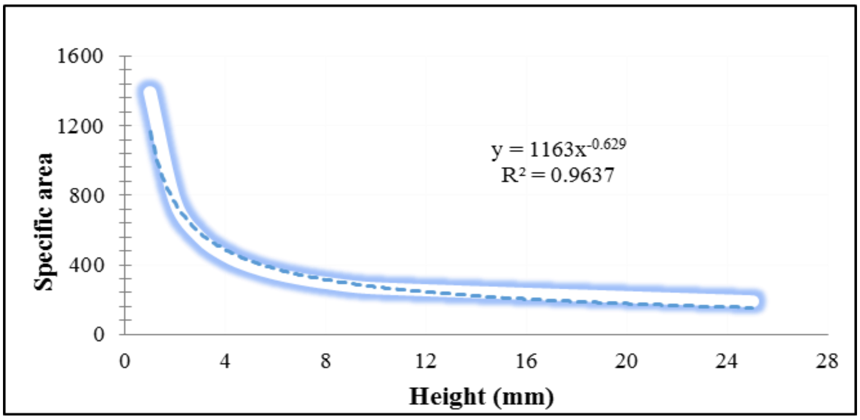


Fig. 14. Plot analysing chip length vs specific area

2.5 Designing the UV Treatment System

Designing the UV treatment system required background knowledge and the use of continuous flow operation modelling with cell death factor in. Assuming there is no axial dispersion, the time the process needs to take for sterilisation is given by Eq. (29) [11].

$$Time\ for\ sterilisation = \frac{ln^{Ni}/_{Nf}}{k_d} \tag{29}$$

where N_i and N_f are the number of cells that are present initially and the number of cells present in the end, respectively. Found the speed of the flowrate through the chamber by Eq. (30).

$$linear\ velocity = \frac{Flowrate}{Cross\ Section\ Area} \tag{30}$$

Since the diameter of the pipe is 2 inch, or 0.05 m, convert the 160 L/min to appropriate units and get a linear velocity of 0.816 m/h. Using sterilisation values of cell death rate from literature, k_d is 3.6 h^{-1} [11]. Then got the time of sterilisation 0.748 h^{-1}. Multiplying this time by the linear velocity should give a length for the UV treatment chamber gives Eq. (31).

$$Length = Velocity\ x\ Time = 0.816\ x\ 0.748 = 0.613\ metres \tag{31}$$

This length translated to roughly below 2 feet. In the interest of over designing, to make sure the chamber was over 2 feet long.

2.6 Fabrication of Flocculation Tank

This tank is used for coagulation and flocs formation by the addition of coagulant chemical (alum) and with continuous low revolution stirring. Coagulant discharges the particles and lets them physically combine on collision to form large particles flocs that settle faster.

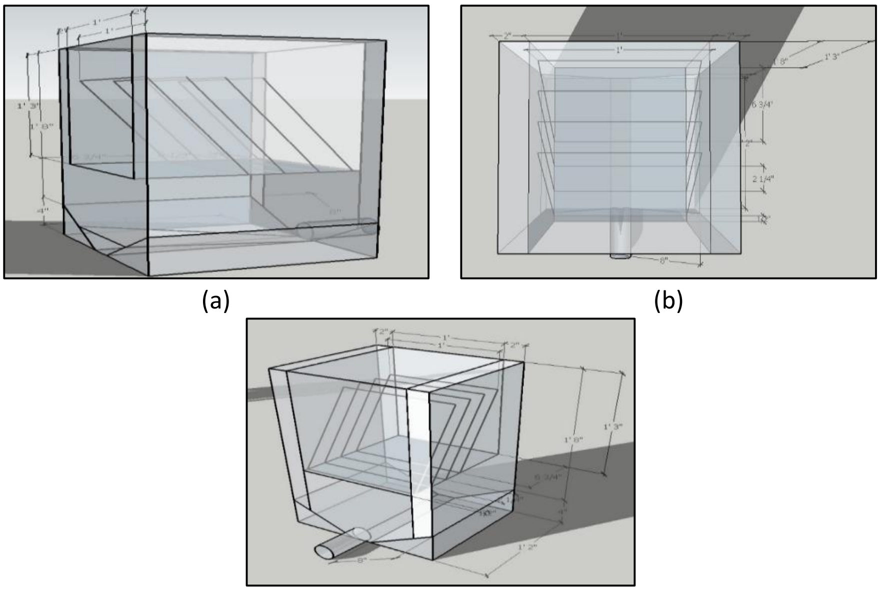
Commercially available circular barrel type tank of 60 litres volume has opted for this application. Post modifications have been performed by making inlet and outlet flow connections, along with the provision of agitator shaft in the top cap. Figure 15 presents the gear motor of 12 V with dual propeller type agitator on a shaft of 1 m length has been mounted, and 60 RPM of the motor has been set for optimum flocculation based on the literature review. It has been successfully tested at the 85% fill volume of around 52 litres.

(a) (b)

Fig. 15. Flocculation tank (a) gear motor for continuous stirring at 60 RPM (b) Impellers at an angle

2.7 Fabrication of Lamella Tank

The important considerations that have been taken into account for fabricating lamella tank are structural robustness, light weightiness, water inertness and internal flow visibility. Based on the above-mentioned considerations, transparent acrylic plastic sheets of 4 mm thickness have opted as the material of construction for the lamella tank. The 3-D software generated preliminary design views along with their respective dimensions are shown in Figure 16.



(a) (b)

(c)

Fig. 16. Design of flocculation tank (a) perspective view of the tank (b) top view of tank and (c) isometric view of tank

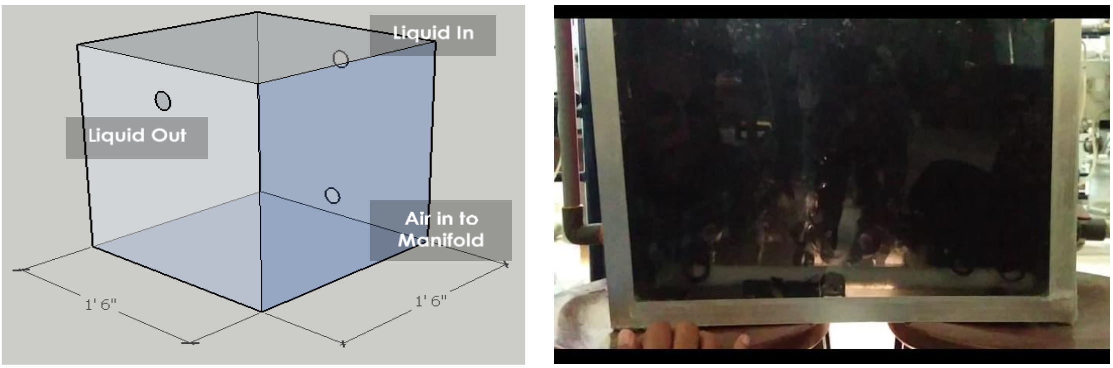
The actual complete fabricated Lamella tank views are shown in Figure 17.

(a) (b)

Fig. 17. Lamella tank designing (a) lamella sheets being constructed (b) complete Lamella Tank

2.8 Fabrication of the Moving Bed Bio Reactor (MBBR)

The design of the MBBR tank needed to be robust and able to withhold the continuous vibrations of the aeration process that takes place during the BOD reduction. Also, since the tank consists of thousands of moving particles, they are bound to crash into and bounce off the walls of the tank, so the walls have to be scratch resistant. The tank was built out of a combination of materials, namely 4mm clear acrylic sheets and 1.3 mm fiber plastic sheets. The fiber plastic sheets are opaque. They were chosen over a complete acrylic sheet tank due to their increased durability and ability to withstand scratches and wear and tear. Since the sheets are made of fiber reinforced plastic, they have good strength in all directions and are strong under tension and compression. The sheets have a high elastic limit and toughness, thereby allowing them to withstand the effects of buckling that might occur in a tank filled with liquid. A 3D model of the tank was made before actual construction. Based on a volume of around 50 L, the tank was made with dimensions of 1.5 ft. on each side, which gave it a volume of 3.375 ft^3. This translates to a liquid holding volume of around 95 litres if filled to the brim. A simple view of the tank is shown in Figure 18.



(a) (b)

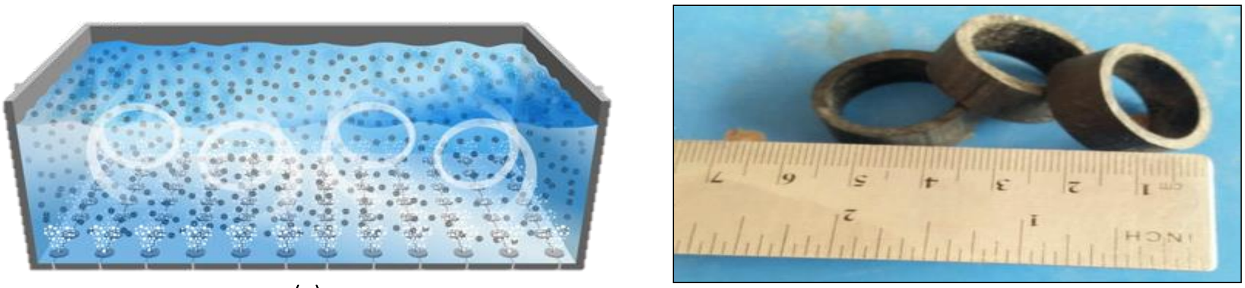
Fig. 18. MBBR tank designing (a) 3D view of MBBR design showing inlet and outlet the aeration (b) constructed MBBR in operation

The value of 95 L is almost double of the required liquid volume. This is by design because space was left at the top of the tank for connections and pipes as shown in Figure 19. Also, since treating wastewater usually produces a lot of foam, space had to be left for the volume that it would occupy.

Fig. 19. View from above, turbulent and aerated top surface

Figure 20 shows the chips that were designed earlier had to be made of a very specific material. The material had to be less dense than water. This would allow the chips to float in the water and be circulated the liquid by the aeration.

The chips also had to be non-toxic to the bacteria and unreactive to oxygen. They had to be resistant to wear and tear and cheap enough to mass produce. A lot of options were looked at and after careful analysis of relative densities, costs, availabilities and inertness, finally chose to make the chips out of High-Density Poly-Ethylene (HDPE). HDPE has a density of around 0.91 kg/m^3 and is very resistant to chemical attack. It is not very rigid, but it makes up for it by being very resistant to wear and tear [21].



(a) (b)

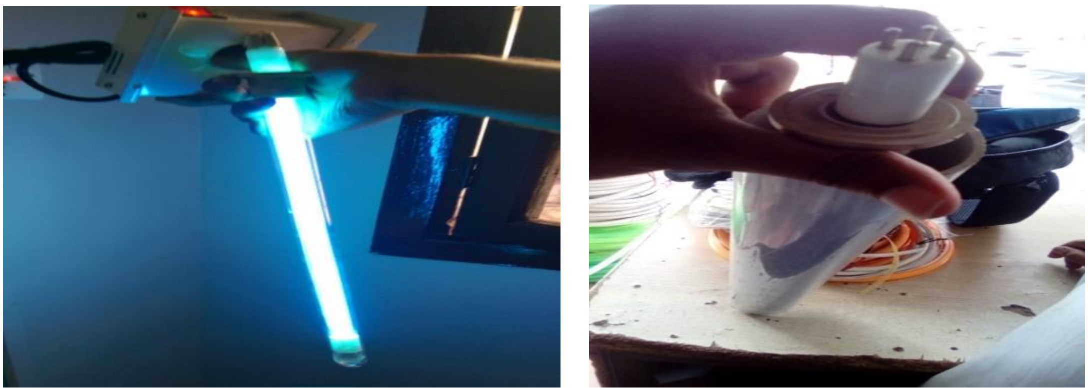
Fig. 20. Chip pattern (a) expected flow pattern of chips, (b) example of Chips that were designed and used

2.9 Fabrication of UV Treatment System

According to the spatial constraint, designed UV treatment equipment in such a manner that it would occupy the least space possible as display in Figure 21. The exposure time of UV is extremely low. Around 20-30 seconds, so it meant that not a lot of area was required. Hence, keeping in mind all the factors a 20-inch UV bulb was procured. The wavelength emitted by this medium pressure mercury bulb falls between the ranges of 250-270 nm. Which is required to kills the bacteria, viruses and protozoa. The warranted life of the bulb is 4000-8000 hrs. The UV bulb rating is 15 watts. The UV tube came with a power supply of 24 volts as default was 12 volts. A significant challenge was the selection of UV tube sleeve material. With a lot of materials available in market stainless steel pipe was the best option for sleeve material but it was too expensive. So, the next best option was PVC material for the sleeve as it is easily available. However, the main problem that arises by using PVC with UV is that it degrades PVC pipe. Continuous exposure to UV causes only the exposed surface to change its molecular structure. The reason for this molecular change is the energy that is emitted by the UV and absorbed by the PVC which causes the electrons to become excited. Due to this degradation, the PVC site which is exposed becomes yellow in colour and brittle. The reason to go with PVC material is that although it is degraded by UV, it does not add any harmful toxins into the water stream and the composition of water is unaltered [6]. As this project is a prototype and using it does not affect the overall performance of the system using PVC for the sleeve material is not a grave issue. The 23-inch PVC sleeve for UV tube was designed with an inlet at one end and an outlet on the other end. Inside the 23-inch PVC sleeve, the 20-inch UV tube was fixed along with the connections to the 24 volts' power supply. The list of materials used are given in Table 9.

Table 9
List of materials used

Serial No.	Name	
1	Acrylic sheets (4mm)	Aluminum Angles
2	Fiber plastic sheets	Aluminum sheets
3	Valves	Rubber pipes
4	Compressor	Geared motor
5	PVC	Iron stirrer
6	Iron	Power supplies
7	Silicon	Biofilm carrier chips
8	UV tube 20-inch	Epoxy
9	Coagulant (Alum)	



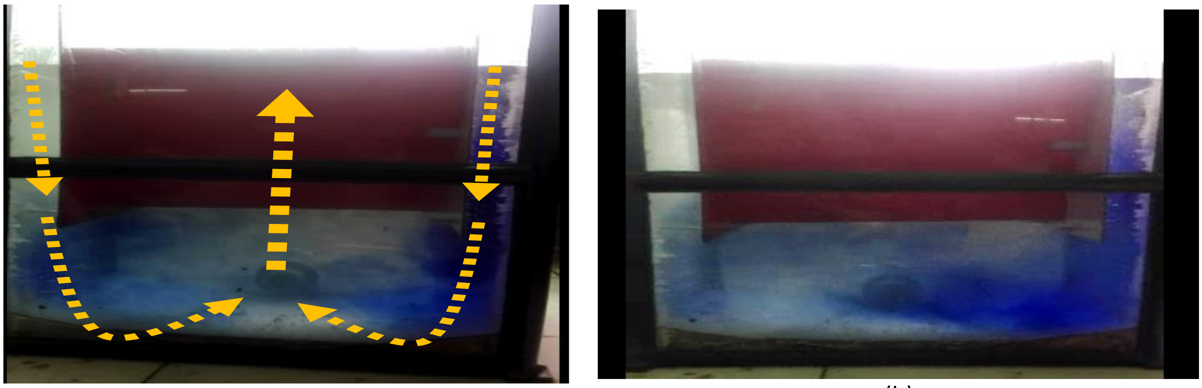
(a) (b)

Fig. 21. UV designing (a) 20inch UV tube along with 24V power supply, (b) PVC sleeve of 23-inch in which UV tube is fitted

3. Results

3.1 Results from Lamella Settling Tank

Two types of tests have been performed on the lamella settling tank. Initially preliminary "Ink Tests" have been executed on the tank to observe the flow pattern in order to evaluate the settling effectiveness. Commercially available blue ink solution has been made and combined with water inflow, ink dispersion has been observed with the continuous flow. Figure 22 shows the prevailing flow pattern in the tank.



(a) (b)

Fig. 22. Ink test (a) ink test in progress showing flow pattern of settling tank (b) settling being tested via ink test

From the successful execution and observations of ink test, it can be concluded that they are no by-passing, and uniform flow distribution of flow over plates which is amongst the most important characteristic for the effective performance of lamella tank [17, 19]. After the success of ink test for making sure the desired flow patterns, series of sewage water settling experiments have been performed on the lamella tank. Flocculated water has been charged and experimentation conducted for 2 hours of continuous operation. Initial turbidity and final, after 2 hours, turbidity have been measured from the collected samples and the results are shown in Table 10. The obtained results are very promising and are in good agreement with the required selected standard value of under 5 NTU.

Table 10
Turbidity test results from settling tank

	Initial Turbidity	Final Turbidity
Test 1	9.1	4.1
Test 2	9.4	5.0
Average Result	9.25	4.55

3.2 Results From MBBR

Once the chips had been added to the reactor, it was just a matter of finessing the flowrate of the air that was needed to create a turbulent layer in the tank. The turbulent layer was needed so that the less dense chips could circulate evenly and proper mixing could be achieved without a stirrer. Though designing the MBBR was hard, the tests were simple and easy to carry out. Partially treated sewage water from the settling tank was taken and placed into the MBBR and run in batch cycles of 7 hours and 22 minutes, a time that was calculated before. Samples of the water before the process was begun and after the process has begun were taken. The results of BOD before and after running MBBR are tabulated in Table 11.

Table 11
Results of BOD_5 before and after running MBBR

	Initial BOD	Final BOD
Test 1	220	43
Test 2	230	27
Average Result	225	35

When all the equipment was up individually, the components were tested for the function they were created and the results were tabulated. A final integrated test was run as well, with the water sample being tested. A final result is shown in Table 12 for comparison.

Table 12
Comparison of results

	Unit	Raw Sewage	Limit by Standard	Post Treatment
TDS	mg/L	800	1000	753
Electrical Conductivity	dS	0.89	1.50	0.91
pH	-	6.9	6.5 – 8.5	7.6
BOD	mg/L	220	8	20
Ammonia	mg/L	1.7	1.0	1.1
Fecal Coliform	CFU/100ml	15000	1000	265

The selected parameters are set to achieve and were unable to get the desired values on all of the tests. The biggest cause of concern was the BOD value, which were unable to reduce to the amount of 8 mg/L that is set by the standard. However, this was not without a fix and the root cause was identified as differences between chip numbers needed in ideal conditions compared to those needed in reality. Increasing the number of chips that had in the Bioreactor should give much closer results. The settling tank after the MBBR had limitations in creating anoxic regions and thus did not allow for proper degradation of nitrogenous based compounds [20,23]. The TDS was a major concern, but due to multiple tests confirming that the TDS level never rose more than 800 mg/L, it never became a parameter that needed to address and reduced the complexity of the system. It remains a system that can be added inline if the feed concentrations change. Due to the oversizing of UV treatment, and the high residence time afforded to the water, the sterilisation achieved was quite high, making the water biologically safe to use and free of smell.

3.3 Operational Cost Analysis

The feasibility and applicability of any project depend on the cost analysis. In this project per unit cost of treated wastewater is an important aspect to be analysed. Cost analysis includes both the capital and operational cost determination. However, the capital cost of the process equipment is along with all auxiliaries of the developed prototype is an ambiguous or rather a misrepresentative part because, from the prototype equipment’s material selection to the fabrication processes employed, depends on a budget of the group. Therefore, this is a very subjective matter and to give an accurate capital cost figure is no more a relevant matter. On the other hand, the operational cost can be analysed based on the electricity consuming equipment’s and added chemicals employment costs. It consists of two parts, continuously added chemicals cost and running equipment’s electricity cost [27].

The only chemical used in the whole treatment process is alum as a coagulant. Selected alum concentration is 100 mg/liter. For a flow rate of 160 litres/day capacity, alum requirement is given by Eq. (32).

$$Alum\ requirment = \left(100\,\frac{\text{mg}}{\text{liter}}\right) * \left(160\,\frac{\text{liters}}{\text{day}}\right) = 16\,\frac{\text{gm}}{\text{day}} \tag{32}$$

From the market survey, alum is available at 200 Rs/Kg in Karachi. Therefore, alum daily cost is given by Eq. (33).

$$Alum\ daily\ cost = \left(16\frac{\text{gm}}{\text{day}}\right) * \left(200\frac{Rs}{1000gm}\right) = 3.2\frac{Rs}{day} \tag{33}$$

Hence, it costs around 3.2 Rs/day for 160 litre/day capacity. There are three pieces of equipment currently employed in the whole process that uses electricity; flocculation tank stirrer, UV-light as a disinfectant for tertiary treatment and air compressor for aeration. The unit of electricity is needed to compute the operating cost of this equipment. According to K-Electric billing policy, per unit cost varies from billing units. However, an average unit cost is sufficient to estimate the cost requirements. Average unit cost is taken to be 4 Rs/Kwh. Therefore, all calculations have been computed with this rate. One gear motor of 12 V and 3.5 A power rating has been used for the required 30-40 rpm for stirrer tank. Daily power requirement is calculated by Eq. (34).

$$Power\ requirement = (12\ Volts) * (3.5\ Amp) = 42\ Watts \tag{34}$$

Therefore, daily cost is given by Eq. (35).

$$Operating\ Cost = (42\ Watts) * \left(24\frac{hours}{day}\right) * \left(4\frac{Rs}{Kwh}\right) = 4.03\frac{Rs}{day} \tag{35}$$

Hence, it costs around 4 Rs/day for 160 liter/day capacity.

UV-Light as a disinfectant for tertiary treatment, a bulb of 20 inches UV-Light has been used in the process. Its power rating is 15 Watts. Therefore, daily cost is given by Eq. (36).

$$Operating\ Cost\ (UV) = (15\ Watts) * \left(24\frac{hours}{day}\right) * \left(4\frac{Rs}{Kwh}\right) = \ 1.44\frac{Rs}{day} \tag{36}$$

Hence, it costs around 1.44 Rs/day for 160 litres/day capacity.

Although the compressor that could satisfy the process needs has the following specifications;

Reciprocating PD-Compressor = 0.5 Hp motor with 40 GPM maximum capacity. Therefore, operational cost for compressor operating cost can be estimated as Eq. (37) [27]:

$$Operatoional\ Cost = (0.5\ \ hp) * \left(746\frac{watts}{hp}\right) * \left(24\frac{hours}{day}\right) * \left(4\frac{Rs}{Kwh}\right) = 36\frac{Rs}{day} \tag{37}$$

Hence, it costs around 36 Rs/day for 160 litres/day capacity.

From the above calculated individual cost estimations, total operational cost per day for 160 litres/day capacity can be computed as Eq. (38) [3].

$$Total\ Operational\ Cost = Chemical\ Costs + Electricity\ Cost \tag{38}$$

The operating cost per day 44.62 Rs/day.

Therefore, operational cost per litre of treated water is given by Eq. (39).

$$Operational\ Cost\ per\ Liter = \frac{44.62\frac{Rs}{day}}{160\frac{liters}{day}} = 0.278\frac{Rs}{litre} \tag{39}$$

4. Conclusions

The simplification of the problem has been rectified for a raw sewage water, treated and convert it into the usable water in a domestic, spatially constrained environment. The goal was accomplished to give a solution to the thousands of city dwellers who had exceptionally constrained access to fresh water but had space and resources to treat their wastewater in usable water and in this manner recover a portion of the water that would some way or another be disposed of as waste. A prototype consisting of various, state of the skill settling tanks, bio reactors and sterilization units was assembled and a continuous treatment process was put together. This technique decreases the time and the space taken to treat water by numerous components with the goal that a single unit may fit in a small apartment as well, focusing on the individuals who have least amount of resources to settle their water issue. The feasibility and applicability of operational cost per litre of treated water by this method is 0.278 Rs/litre.

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