APPLICATION OF MEMBRANE BIOREACTOR FOR TEXTILE WASTEWATER TREATMENT: PILOT PLANT PROCESS MODELLING AND SCALE-UP

1.1) NECESSITY FOR MBR PRIMARY TREATMENT OF TEXTILE EFFLUENT

Pilot plant is important in the sense that it drastically improves our understanding of the functioning of the system for the given raw effluent characteristics and environmental conditions. It also helps to scale-up the results and optimize the size of the full-scale plant. In this respect, it is important that pilot plant tests shall be conducted before implementation of full-scale plant. This article describes how a MBR pilot plant tests could be modelled based on the raw effluent characteristics, and the modelling results could be utilized for scaling-up.

The selection of MBR for textile waste water treatment is justified on the following lines:

1) Membrane Bioreactor is an advancement over the conventional activated sludge process with the use of ultrafiltration or, microfiltration membrane, which helps to maintain high levels of MLSS concentration and better treated water quality.
2) Activated sludge process has been well documented, thoroughly experimented, and widely adopted for the treatment of industrial waste water, including that of textile waste water.
3) While conventional biological treatment systems may face problems due to:
   a. Low F/M ratio (due to low levels of BOD in the raw effluent);
   b. Sludge settleability in the clarifier;
   c. Growth of algae due to higher levels of total dissolved solids in the raw effluent;
   d. Increased operating costs due to low levels of MLSS concentration, increased aeration tank size, requirement for higher retention time;
   e. Inability to withstand *shocks*, *i.e.*, sudden changes in the raw effluent quality;
   f. Requirement for more secured land fill area due to higher levels of sludge production;
   g. Inefficient nutrient removal in the raw effluent that shall encourage growth of microbial organisms on the reverse osmosis membrane;
   h. Poor treated water output quality which shall impose operational restrictions on the secondary treatment system,

1.2 MODEL CALCULATIONS FOR PLANT DESIGN

1.2.1 Design principles

The pilot plant should simulate the full-scale plant. Considering that the adopted full-scale plant implements MBR with step-feed anoxic/aerobic system[1], the pilot plant configuration shall also follow the same line. In this regard, a single step-feed anoxic/aerobic design for the pilot plant shall be employed. This means that there shall one anoxic chamber followed by one aerobic chamber and a hollow fibre membrane chamber similar to the full-scale plant. The mathematical modelling of the biological processes for determination of various system parameters are described below in detail[2].

---

1 Any redox reaction involves an electron donor - electron acceptor pair. As per definition, the electron donor is *oxidized* while electron acceptor is *reduced*. Generally, the terms anaerobic and anoxic are used interchangeably to represent the absence of oxygen in the waste water. However, there is an important difference as outlined here. Under biological oxidation/reduction processes, the terms anaerobic and anoxic have different notions in the sense that *anaerobic* refers to the general condition where dissolved oxygen is absent in the effluent, and any one or more of electron acceptors present in the waste water such as nitrate, nitrite and sulphate can act as electron acceptors. The term *anoxic* refers to the condition where only nitrate/nitrite are the only electron acceptors taking part in the biological oxidation/reduction processes. Some bacteria can use oxygen as electron acceptor when it is available, and in the absence of oxygen the same bacteria can switch the respiration mode to utilize nitrate/nitrite as electron acceptors. These kinds of bacteria are called *facultative bacteria*. These kinds of bacteria can survive both in anoxic as well as in aerobic conditions unlike other kinds of bacteria that can survive only in anaerobic or aerobic conditions. As MBR configuration involves anoxic, aerobic, and membrane compartments with re-circulation from the membrane zone to the anoxic zone, the anoxic zone shall have very low levels of dissolved oxygen (the condition chemically referred as sub-oxic) brought back by the recirculating effluent. Thus, facultative bacteria only dominate the scene in the MBR. This technically means that in the anoxic compartment, denitrifying bacteria shall dominate while, in the aerobic chamber other aerobic bacteria shall also present but with the dominance of facultative bacteria. This is an important factor to be considered while considering mathematical modelling of biological processes in the MBR. The adopted configuration is meant to remove nitrogen in the anoxic compartment, and BOD in the aerobic compartment. Thus, step-feed anoxic/aerobic system considered here is suitable for modelling the adopted MBR process.

2 For further discussion please refer Metcalf & Eddy (2003) Wastewater Engineering: Treatment and Reuse (Fourth edition), Tata-McGraw Hill Publishing Company Limited, New Delhi.

1.2.2 Design approach

The modelling design adopts complete-mix activated sludge treatment process for BOD removal[3]. The textile waste water quality utilized for modelling was obtained through analysis of the raw effluent (Table-1.1), and also calculated using necessary equations. In order to design the MBR, the values of kinetic coefficients (refer Table-1.2[4]) have to be obtained through pilot plant test data and parameter estimation through computer aided simulation. In the absence of pilot plant test results, the design approach here is to adopt recommended values of parameters for industrial waste water treatment (listed in Table-1.2), and obtain full-scale plant design details that approaches the design with actual pilot plant test results.

Table-1.1: Textile waste water quality utilized for modelling

S.No.	*Parameter*	*Value*	*Unit*
1	BOD	261.2	g/m^3
2	sBOD (soluble BOD)	190.6	g/m^3
3	COD	545.5	g/m^3
4	sCOD (soluble COD)	398.2	g/m^3
5	TSS	660	g/m^3
6	VSS	396.7	g/m^3
7	TKN	30	g/m^3
8	NH_4-N	2.3	g/m^3
9	Alkalinity	855	g/m^3
10	bCOD/BOD ratio	1.6	

3 Most exhaustive computer aided simulations for parameter estimation are carried out with the use of ASM1 model of the International Water Association. For this purpose, WEST and AQUASIM computer programmes were developed and demo versions of the software are available over the Internet. This model has been advanced into ASM2 and ASM3. For all practical purposes of scaling up pilot plant test results, the complete-mix activated sludge treatment process described here shall be enough. In the present model, nitrification and denitrification were not considered as the main objective is to address COD/BOD reduction. Further, introduction of nitrification/denitrification does not alter the model and estimated values significantly.

4 Adopted from Metcalf & Eddy (2003) Wastewater Engineering: Treatment and Reuse (Fourth edition), Tata-McGraw Hill Publishing Company Limited, New Delhi.

1.2.3 Design conditions and assumptions

1) Fine bubble diffuser with an aeration clean water O_2 transfer efficiency = 28%.
2) Liquid depth for the aeration basin 5 *m*.
3) The point of air release for the ceramic difference is 0.5 *m* above the tank bottom.
4) DO in aeration basin = 2 g/m^3.
5) Site elevation is 304.2 *m* above MSL (988 ft)
6) Aeration α = 0.45 for BOD removal; β = 0.95, and diffuser fouling factor F = 0.90.
7) The kinetic coefficients are as given in Table-1.2
8) SRT for BOD removal = 20 days
9) Design MLSS XTSS concentration = 10,000 g/m^3
10) TKN peak/average factor of safety FS = 1.5

Table-1.2: Activated sludge kinetic coefficients for heterotrophic bacteria at 20°C[5].

Parameter	*Coefficient*	*Unit*	*Range*		*Typical value*
Maximum specific growth rate	μ_m	g VSS/g VSS . d	3.00	13.20	6.00[6]
half-velocity constant	K_s	g bCOD/m^3	5.00	40.00	20.00
Biomass yield, mass of cell formed per mass of substrate consumed	Y	g VSS/g bCOD	0.30	0.50	0.40
Endogenous decay coefficient	k_d	g VSS/g VSS . d	0.06	0.20	0.12
Fraction of cell mass remaining as cell debris	f_d	Unit less	0.08	0.20	0.15
	θ values				
	μ_m	Unit less	1.03	1.08	1.07
	k_d	Unit less	1.03	1.08	1.04
	K_s	Unit less	1.00	1.00	1.00

5 The values are converted to 30°C using appropriate equations where ever necessary.
6 A value of 5.0 was adopted for the calculation based on advice from the supplier of MBR.

To find the values of μ_m, kd, Ks at a temperature different from 20°C,					

1.2.4 Design parameters

The following parameters shall be considered for designing calculations. Some of them require analysis of the effluent, while many of them could be determined using applicable equations. In the following Table-1.3, the list of all parameters and their values are provided.

Table-1.3: Design parameters for modelling calculations

S.No.	*Parameter*	*Notation*	*Value*	*Unit*
1	Biodegradable COD	bCOD	Estimated in the model	g/m^3
2	Non-biodegradable COD	nbCOD	Estimated in the model	g/m^3
3	Effluent soluble COD	sCODe	Estimated in the model	g/m^3
4	Non-biodegradable VSS	nbVSS	Estimated in the model	g/m^3
5	Biodegradable particulate COD	bpCOD	Estimated in the model	g/m^3
6	Particulate COD	pCOD	Estimated in the model	g/m^3
7	Soluble BOD	sBOD	Analysed value (raw effluent)	g/m^3
8	Soluble COD	sCOD	Through raw effluent analysis	g/m^3
9	Inert TSS	iTSS	Estimated in the model	g/m^3
10	Quantity of waste water feed	Q	1500 m^3/day	m^3/day
11	Biomass yield	Y	Estimated in the model	g VSS/g bCOD
12	Influent BOD or, bsCOD concentration	S_o	Analysed value (raw effluent)	g/m^3
13	Concentration of growth limiting substrate in solution	S	Estimated in the model	g/m^3
14	Half-velocity constant	K_s	Recommended value	g/m^3
15	Endogenous decay coefficient	k_d	Recommended value	day^{-1}
16	Solid Retention Time	SRT	20 days	day

Table-1.3: Design parameters for modelling calculations				
17	Maximum specific growth rate	μ_m	Recommended value	g VSS/g VSS.d
18	Mass of VSS in the aeration tank	$P_{X,VSS}$		
19	Mass of TSS in the aeration tank	$P_{X,TSS}$		
20	Biomass as VSS wasted per day	$P_{X,bio}$		Kg/d
21	Endogenous decay coefficient for nitrifying organisms	k_{dn}		gVSS/g VSS.d
22	Concentration of NH_4-N in the influent flow that is nitrified	NOx		g/m^3
23	Influent waste water TSS concentration	TSS_o		g/m^3
24	Influent waste water VSS concentration	VSS_o		g/m^3
25	Net waste activated sludge produced each day, measured in terms of total suspended solids	$P_{X,TSS}$		kg/day
26	Amount of VSS produced and wasted daily	$P_{X,VSS}$		kg/day
27	Effluent NH_4-N concentration	N_e		g/m^3
28	Influent TKN concentration	TKN_o		g/m^3
29	Nitrogen oxidized	NO_x		g/m^3
30	Soluble substrate utilization rate	r_{su}		g/m^3.day
31	Mixed liquor biomass concentration in the aeration tank	X		g/m^3
32	Aeration tank volume	V		m^3
33	Food to biomass ratio	F/M		bs COD / g VSS.d or, g BOD/g VSS.d
34	Hydraulic retention time	τ		day
35	Fraction of cell mass remaining as cell debris	f_d		g/g
36	Influent waste water TSS concentration	TSS_o		g/m^3
37	Influent waste water VSS concentration	VSS_o		g/m^3

1.2.5 Equations and parameter relationships

Table-1.4 provides necessary equations and parameter relationships that are utilized in the model calculations.

Table-1.4: Equations and parameter relationships	
1	TKN (Total Kjeldahl Nitrogen) = organic + ammonia nitrogen [TKN g/m^3]
2	$P_{X,bio}$ (= $P_{X,VSS}$)
3	1/SRT = μ (specific biomass growth rate) = $Y (r_{su}/X) - k_d$
4	Mass of MLSS = (X_{TSS}) (V) = $(P_{X,TSS})$ SRT
5	Mass of MLVSS = (X_{VSS}) = $(P_{X,VSS})$ SRT
6	VSS/TSS = 0.601
7	F/M = (Total applied substrate rate/Total microbial biomass) = QS_o/VX
8	F/M = S_o/τ X [Q/V = τ]
9	$P_{X,VSS} = \dfrac{QY (S_o-S) (1\ kg/ 10^3\ g)}{[1 + (k_d)\ SRT]} + \dfrac{(f_d) (k_d)\ QY(S_o-S)\ SRT\ (1\ kg/ 10^3\ g)}{[1 + (k_d)\ SRT]} + \dfrac{QY_n (NO_x) (1\ kg/ 10^3\ g)}{[1 + (k_{dn})\ SRT]} + Q\ (nbVSS) (1\ kg/ 10^3\ g)$
10	bCOD = 1.6 (BOD) (g/m^3)
11	nbCOD = COD – bCOD (g/m^3)
12	sCODe = sCOD – 1.6 (sBOD) (g/m^3)
13	nbVSS = (1-bpCOD/pCOD) VSS (g/m^3)
14	$\dfrac{bpCOD}{pCOD} = \dfrac{(bCOD/BOD)\ (BOD - sBOD)}{(COD - sCOD)}$

15	iTSS = TSS – VSS [Note: VSS/TSS = 0.85 (range: 0.80 – 0.90)]
16	τ = (V/Q) (day)
17	nbVSS = $\{$1-(bpCOD ÷ pCOD)$\}$ VSS
18	[bpCOD ÷ pCOD] = $\big($(bCOD ÷ BOD) (BOD – sBOD)$\big)$ ÷ (COD-sCOD)
19	θ = Temperature activity coefficient.
20	Design SRT = 20 days.
21	$R_o = Q (S_o – S) – 1.42 P_{X,bio}$

1.2.6 Steps involved in the design calculations

1) Development of wastewater characteristics needed for design.

a) Finding bCOD:

bCOD = 1.6 (BOD) = 1.6 (261.2 g/m^3) = 417.9 (g/m^3)

b) Finding nbCOD (non-biodegradable COD):

nbCOD = COD – bCOD = (545.5 – 417.9) g/m^3 = 127.6 (g/m^3)

c) Finding effluent sCODe (assumed to be non-biodegradable):

sCODe = sCOD – 1.6 (sBOD) = 398.2 - 1.6 (190.6) g/m^3 = 93.24 (g/m^3)

d) Finding nbVSS (non-biodegradable VSS)

nbVSS = (1-bpCOD/pCOD) VSS (g/m^3)

$$\frac{bpCOD}{pCOD} = \frac{(bCOD/BOD)\ (BOD\text{-}sBOD)}{(COD - sCOD)} = \frac{(417.9/261.2)\ (261.2\text{-}190.6)}{(545.5 - 398.2)} = \frac{112.95}{147.30} = 0.7668\ (g/m^3)$$

e) Finding the iTSS (inert TSS)

iTSS = TSS – VSS (g/m^3) = (660 – 396.7) = 263.3 (g/m^3)

2) Designing suspended growth system for BOD removal only.

a) Determining biomass production:

$$P_{x,vss} = \frac{QY\ (S_o\text{-}S)}{(1 + k_d\ SRT)} + \frac{f_d\ k_d\ QY\ (S_o - S)\ SRT}{(1 + k_d\ SRT)}$$

Defining input data for above equation:

Q = 1500 m^3/day.
Y = 0.453 g VSS/g bCOD
S_o = 417.6 g bCOD/m^3
S = effluent substrate concentration (g bCOD/m^3)

Calculation of effluent substrate concentration (S):

$$S = \frac{K_s\ (1 + (kd)\ SRT}{SRT\ (\mu_m - kd) - 1}$$

K_s = 20 g/m^3 (Typical value from <u>Table-1.2</u>)

$\mu_{m,T} = \mu_m \theta^{(T-20)}$
$\mu_{m,T} = (5.0\ g/g\ .d)\ (1.07)^{(30-20)} = 9.84\ g/g.d$
$k_{d,T} = k_{20}\ \theta^{(T-20)}$
$k_{d,30} = (0.12\ g/g.\ d)\ (1.04)^{(30-20)} = 0.18\ g/g.\ d$

Calculating S:

$(20\ g/m^3)\ [1 + (0.18\ g/g.d)\ (20\ d)] \div \{(20\ d)\ [9.84 - 0.18)\ g/g.d - 1]\}$
$92 \div 173.2 = 0.53\ g\ bCOD/m^3$

Solving for $P_{X,VSS}$

$$P_{X,VSS} = \frac{(1500)\ (0.453)\ (417.9 - 0.53)\ (1\ kg/\ 10^3\ g)}{[1 + (0.18)\ (20)\ ]}$$

$$+ \frac{(0.15)\ (0.18)\ (1500)\ (0.453)\ (417.9 - 0.53)\ (20)\ (1\ kg/\ 10^3\ g)}{[1 + (0.18)\ (20)]}$$

$= 61.61 + 33.27 = 94.88$ (kg/day)

3) Determining the mass of VSS and TSS in the aeration basin.

a) Determination of PX,VSS and PX,TSS:

Mass = P_x (SRT)
$P_{x,VSS}$ = 94.88 kg/d + Q (nbVSS) g/day = 94.88 + [(1500) (0.7668) (1 kg/10^3 g) = 1.15 kg/day] = 96.03 kg/day
$P_{X,TSS}$ = (94.88 kg/day)/0.601 + 1.15 kg/day + Q ($TSS_o - VSS_o$)
= 157.87 + 1.15 + [1500 (660 – 396.7)] = 553.97 kg/day

b) Calculating the mass of VSS and TSS in the aeration basin.

i) $(X_{VSS})(V) = (P_{X,VSS})\ SRT$

$= (96.03\ kg/day)\ (20\ day) = 1920.60\ kg$

ii) $(X_{TSS})(V) = (P_{X,TSS})\ SRT$

$= (553.97\ kg/day)\ (20\ days) = 11079.40\ kg$

4) Selecting a design MLSS concentration and determination of the aeration tank volume and detention time using the TSS mass computed in step (3).

a) Determining the aeration tank volume using the relationship from step-(3b).

$(V)(X_{TSS}) = 11079.40\ kg$

At $X_{TSS} = 10{,}000\ g/m^3$

$V = [(36{,}930.8\ kg)\ (10^3\ g/kg)] \div 10{,}000\ (g/m^3) = 1107.94\ m^3$

b) Determining the aeration tank detention time

$\tau = V/Q = (1107.94\ m^3)\ (24\ h/day) \div (1500\ m^3/day) = 17.73\ h$

c) Determination of MLVSS:

Fraction VSS = 1920.60 kg VSS ÷ 11079.40 kg TSS = 0.17

MLVSS = 0.17 (10,000 g/m^3) = 1733 g/m^3

5) Determination of F/M and BOD volumetric loading.

a) <u>Determining F/M ratio:</u>

F/M = (QS_o/XV) = kgBOD ÷ kg MLVSS.d
= [1500 m^3/day) (261.2 g/m^3)] ÷ [(1733 g/m^3) (1107.94 m^3)]
= 0.204 kg/kg.d

b) <u>Determining volumetric BOD loading:</u>

BOD loading = $(QS_o \div V)$ = kg BOD ÷ $m^3.d$
= (1500 m^3/day) (261.2 g/m^3) ÷ (1107.94 m^3) (10^3 g/kg)
= 0.354 $kg/m^3.d$.

6) Determining the observed yield based on TSS and VSS.

a) Observed yield based on TSS

Observed yield = g TSS/g bCOD = kg TSS/kg bCOD
$P_{X,TSS}$ = 553.97 kg/day
bCOD removed = Q $(S_o\text{-}S)$
= (1500 m^3/day) [(417.9 – 0.53) g/m^3] (1 kg/10^3 g)
= 626.05 *kg/d.*

$Y_{obs,TSS}$ = (553.97 kg/d) ÷ (626.05 kg/d) = 0.885 kg TSS/kg bCOD
= (0.885 g TSS/g bCOD)
= (0.885 g TSS/g bCOD) (1.6 g bCOD/g BOD) = 1.416 g TSS/g BOD

b) Observed yield based on VSS

$Y_{obs,VSS}$ = VSS/TSS = 0.601
= (553.97 kg/d) ÷ (626.05 kg/d) = 0.885 g TSS/g bCOD
= (0.885 g TSS/g bCOD) (0.601 g VSS/g TSS)
= 0.532 g VSS/g bCOD
= (0.532 g VSS/g bCOD) (1.6 g bCOD/g BOD)
= 0.851 g VSS/g BOD

7) Calculate the O_2 demand.

Ro = $Q (S_o - S) – 1.42 P_{X,bio}$
= (1500 m^3/day) [(417.9 - 0.53 *g*) *g/m*^{3}] (1 kg/10^3 g) – 1.42 (94.88 kg/d)
= 412.62 kg/d
= 17.193 kg/h

8) Fine bubble aeration design – determining air flow rate at average design flow rate.

$$SOTR = \frac{OTR}{\{[(\beta \otimes C_{ST}) - C_W]/C_{S,20}\} \otimes \alpha \otimes F \otimes (\theta^{(T-20)})}$$

SOTR = Standard oxygen transfer rate (lb O_2/h).
OTR = Amount of oxygen required $(lb/h)_{BOD}$ for oxidation of BOD.
β = The value relating oxygen saturation in waste water compared to clean water – given by the ratio of oxygen saturation in waste water ($C_{s\ waste\ water}$) to oxygen saturation in pure water ($C_{s\ pure\ water}$); a value of 0.95 – 1 is applicable for waste water.
C_{ST} = Oxygen saturation concentration corrected for altitude and temperature (mg/L).
C_W = Operating dissolved oxygen concentration (2-4 mg/L is typical range for waste water plants; 2 mg/L is adopted here).
$C_{S,20}$ = Oxygen saturation concentration for pure water at 20°C (mg/L)

α	=	Ratio of oxygen transfer efficiency (OTE) in waste water to OTE in pure water ($\alpha = 0.45$ for fine bubble diffuser).
F	=	Fouling factor, which accounts for membrane fouling (typically 0.65 to 0.9; a value of 0.9 was adopted for the calculation).
θ	=	Arehenius constant used to correct for the effect of temperature (T) [$\theta = 1.024$ at 20°C][7].

Therefore,

$$SOTR = \frac{37.904\ (lb/h)}{\{[(0.95 \otimes 8.19) - 2]/9.02\} \otimes 0.45 \otimes 0.9 \otimes 1.024^{(30°C-20°C)}}$$

$$= 37.904 \div 0.329$$
$$= 115.21\ lb/h\ (52.258\ kg/h).$$

Converting the SOTR to Standard Cubic Feet per Minute (SCFM) of air required:

$$SCFM = SOTR/(60 \otimes \rho_{air} \otimes SOTE \otimes 0.23)$$

where,

SOTE	=	Standard oxygen transfer efficiency = 0.28 (28% of O_2 actually dissolves into water); this is dependent upon the depth of submergence and the type of diffuser (for diffused aeration system).
ρ_{air}	=	air density (0.073 lb/ft^3 at an altitude of 988 ft and 30°C for Tirupur).
60	=	number of minutes per hour.
0.23	=	pounds of oxygen per pound of air.
SCFM	=	115.21 lb/h/(60 $\otimes$ 0.073 lb/ft^3 $\otimes$ 0.28 $\otimes$ 0.23)
	=	408.442 ft^3/min (11.57 m^3/min)

ρ_{air} can be calculated using the following formula[8]:

7 Effluent temperature at the aeration tank is assumed to be 30°C for the calculations.

8 The air density was found to be 1.164 kg/m^3 (2.57 lb/m^3 or, 0.073 lb/ft^3) for an altitude of 988 ft (301 *m*) for Tirupur, from eXtreme high altitude calculator at: http://bpesoft.com/s/wleizero/xhac/?M=d

ρ_{air} = p.M/(RT)

where,

p = Air pressure at the given altitude and temperature.
M = Molecular weight of dry air (M = 0.0289644 kg/mol).
R = Universal gas constant (8.3145 kPa/K.mol)
T = Temperature in Kelvin (for calculations the ambient temperature of 30°C (303.15 K) is adopted).

The calculated air supply requirement for the MBR is 11.57 m^3/min (694.2 m^3/h).

Authors: S.Eswaramoorthi, K.Dhanapal and D.S.Chauhan. This article has resulted from the earlier work carried out by the first author at EPIC [www.epicin.org], Coimbatore. Your comments and suggestions on this article can be forwarded to: info@ecpconsulting.in Mobile:+91-94439-40818