CHAPTER 5: DESIGN AND CONSTRUCTION OF SEWAGE TREATMENT FACILITIES

5.1   GENERAL

Sewage is 99 % water carrying domestic wastes originating in kitchen, bathing, laundry, urine and night soil. A portion of these goes into solution. The remaining goes into colloidal or suspended stages. It also contains salts used in cooking, sweat, bathing, laundry and urine. It also contains waterborne pathogenic organisms from the night soil of already infected persons. The concentrations are mentioned in Table 5.1

Table 5.1 Contribution of human wastes in grams per capita per day

	Parameters			Range	
1	Biochemical oxygen demand, BOD			45-54	
2	Chemical oxygen demand, COD			1.6-1.9 times BOD	
3	Total organic carbon, TOC			0.6-1.0 times BOD	
4	Total solids, TS			170-220	
5	Suspended solids, SS			70-145	
6	Grit (inorganic,0.2 mm and above)			5-15	
7	Grease			10-30	
8	Alkalinity as calcium carbonate ($CaCO_3$)			20-30	
9	Chlorides			4-8	
10	Total nitrogen N			6-12	
11	Organic nitrogen			~0.4 total N	
12	Free ammonia			~0.6 total N	
13	Nitrate			~0.0-0.5 total N	
14	Total phosphorus			~0.6-4.5	
15	Organic phosphorus			~0.3 total P	
16	Inorganic(ortho- and poly-phosphates)			~0.7 total P	
17	Potassium(as potassium oxide K_2O)			2.0-6.0	
Microorganisms in 100 ml of sewage					
18	Total bacteria	10^9-10^{10}	22	Protozoan cysts	Up to 10^3
19	Coliforms	10^9-10^{10}	23	Helminthic eggs	Up to 10^3
20	Faecal streptococci	10^5-10^6	24	Virus (plaque forming units)	10^2-10^4
21	Salmonella Typhosa	10^1-10^4			

Source: Arceivala, 2000

Note:

1. The wastewater from toilets is usually referred to as black water and the rest of the wastewater from all other activities is referred to as grey water.

2. As already cited in chapter 1, about 12.6 % of population are still not having toilets and practice open defecation. Their grey water somehow gets into sewers by way of open drains discharging into sewers.

3. Thus, the BOD of raw sewage has to be foreseen realistically because this dictates the cost of the STP almost pro-rata.

4. The difference between total solids and suspended solids is the dissolved solids. When calculating its concentration, the dissolved solids in the freshwater used by the ULB must be added to arrive at the values in raw sewage.

5. The raw sewage pH generally ranges between 6.8 to 8.0 depending on raw water quality.

6. The major nitrogen compound in domestic waste is urea $CO(NH_2)_2$, which is readily hydrolyzed to ammonia (NH_3) and carbon dioxide (CO_2) by the enzyme urease present in sewage. Hence, NH_3 constitutes the major fraction of total nitrogen in domestic sewage.

When the treated sewage is discharged into the rivers, the ratio of the respective flows decides the concentration of these parameters in the blended river water. The quality of surface waters for specified uses are shown in Table 5.2

Table 5.2 Use based classification of surface waters in India
(All values are in mg/l unless otherwise specified therein)

Class	Designated best use	Criteria	Limits
A	Drinking water source without conventional treatment but after disinfection	pH	6.5 to 8.5
		Dissolved Oxygen (D O)	6 or more
		BOD	2 or less
		Total Coliform MPN / 100 ml	50 or less
B	Outdoor bathing (organized)	pH	6.5 to 8.5
		Dissolved Oxygen (D O)	5 or more
		BOD	3 or less
		Total Coliform MPN / 1000 ml	50 or less
C	Drinking water source with conventional treatment followed by disinfection	pH	6.5 to 8.5
		Dissolved Oxygen (D O)	4 or more
		BOD	3 or less
		Total Coliform, MPN / 1000 ml	5000 or less
D	Propagation of wild life and fisheries	pH	6.5 to 8.5
		Dissolved Oxygen (D O)	4 or more
		Free Ammonia	1.2 mg/l or less
E	Irrigation, industrial cooling, and controlled waste disposal	pH	6.0 to 8.5
		Electrical Conductivity, micro mhos/cm	< 2250
		Sodium Absorption Ratio (SAR)	< 26
		Boron	< 2 mg/l

Source: The Environment (Protection) Rules, 1986 in http://cpcb.nic.in/GeneralStandards.pdf

The objective of sewage treatment is to reduce the polluting substances to (a) the standards laid down by the Ministry of Environment and Forests (MoEF) of the Government of India (GOI) and these cannot be relaxed by the State Pollution Control Boards (PCB), but they can prescribe more stringent standards specific to the discharge environment and (b) the specified limits of faecal coliforms laid down by the National River Conservation Directorate (NRCD). These standards are compiled and presented in Table 5.3.

Table 5.3 General standards for Discharge of Environmental Pollutants, Part A: Effluents as per Schedule VI of the Environmental (Protection) Rules 1986 and National River Conservation Directorate Guidelines for Faecal Coliforms, (Values in mg/l unless stated)

No	Characteristics	Standards			
		Inland Surface Water	Public Sewers, (A)	Land for Irrigation	Marine Coastal Areas
1	Colour and odour	(B)		(B)	(B)
2	SS	100	600	200	(C), (D)
3	Particle size of SS	(E)	-	-	(F), (G)
4	pH value	5.5 to 9.0			
5	Temperature	(H)	-	-	(H)
6	Oil and grease	10	20	10	10
7	Total residual chlorine	1.0	-	-	1.0
8	Ammoniacal nitrogen (as N)	50	50	-	50
9	Total Kjeldahl Nitrogen, (TKN) (as N)	100	-	-	100
10	Free ammonia (as NH_3)	5.0	-	-	5.0
11	Biochemical Oxygen Demand	30	350	100	100
12	Chemical Oxygen Demand	250	-	-	250
13	Arsenic (as As)	0.2			
14	Mercury (as Hg)	0.01	0.01	-	0.01
15	Lead (as Pb)	0.1	1.0	-	2.0
16	Cadmium (as Cd)	2.0	1.0	-	2.0
17	Hexavalent Chromium (as Cr 6+)	0.1	2.0	-	1.0
18	Total Chromium (as Cr)	2.0	2.0	-	2.0
19	Copper (as Cu)	3.0	3.0	-	3.0
20	Zinc (as Zn)	5.0	15.0	-	15.0
21	Selenium (as Se)	0.05	0.05	-	0.05
22	Nickel (as Ni)	3.0	3.0	-	5.0
23	Cyanide (as CN)	0.2	2.0	0.2	0.2
24	Fluoride (as F)	2.0	15.0	-	15.0
25	Dissolved phosphates (as P)	5.0	-	-	-
26	Sulphide (as S)	2.0	-	-	5.0
27	Phenolic compounds (as C_6H_5OH)	1.0	5.0	-	5.0
	Radioactive materials				
28	Alpha emitters, micro curie/L	10^{-7}	10^{-7}	10^{-8}	10^{-7}
	Beta emitters, micro curie/L	10^{-6}	10^{-6}	10^{-7}	10^{-6}
29	Bio-assay test	(I)			
30	Manganese (as Mn),	2.0	2.0	-	2.0
31	Iron (as Fe),	3.0	3.0	-	3.0
32	Vanadium (as V),	0.2	0.2	-	0.2
33	Nitrate Nitrogen (as N),	10.0	-	-	20.0
34.	Faecal Coliform, MPN/100 ml for discharge	onto land		into water	
		(J)	(K)	(J)	(K)
		1,000	10,000	1,000	10,000

Explanations of notations are given in next page

A. These standards shall be applicable only if such sewer leads to a secondary treatment including biological treatment system; otherwise the discharge into sewers shall be treated as discharge into inland surface waters.

B. All efforts should be made to remove colour & unpleasant odour as far as practicable.

C. For process wastewater 100 mg/l

D. For cooling water effluent 10% above total suspended matter of influent.

E. Shall pass 850 micron IS Sieve

F. Floatable solids max. 3 mm

G. Settleable solids max. 850 microns

H. Shall not exceed 5°C above the receiving water temperature

I. 90 % survival of fish after 96 hours in 100 % effluent

J. Desirable

K. Maximum permissible

Source: The Environment (Protection) Rules, 1986 in http://cpcb.nic.in/GeneralStandards.pdf

In respect of standards specific for treated sewage discharge into surface waters, the values of BOD not exceeding 20 mg/L and SS not exceeding 30 mg/L have been of historical origin. However, this manual recommends the guidelines for treated sewage if discharged into such surface waters used as a source of drinking water as (1) BOD not exceeding 10 mg/L, (2) SS not exceeding 10 mg/L, (3) Total Nitrogen as N not exceeding 10 mg/L, (4) Dissolved Phosphorous as P not exceeding 2 mg/L and (5) Faecal coliforms not exceeding 230 MPN / 100 ml. More details of these can be seen in Table 5-20.

5.1.1 Recommended Exclusive Discharge Guidelines

The recommended guidelines for treated sewage discharge into surface water which after some travel may be used as a source of drinking are mentioned in Table 5-20.

5.1.2 Process of Biological Sewage Treatment

Biological sewage treatment is a process where biological organisms are cultured and allowed to consume the organic matter and multiply their population. This is by a process where the organisms secrete enzymes through their cell walls; and these enzymes solubilize the organic matter and the solution is drawn back into the organisms again through their cell wall. This is the food. The organisms grow and multiply by a process called binary fission whereby each organism splits into two complete new organisms. This is called metabolism. The multiplied organisms are settled out and the clear treated sewage is free from the organic matter. The metabolism can be by (a) aerobic organisms needing oxygen like human beings or (b) anaerobic organisms that do not need oxygen. The pathways are shown in Figure 5.1 and Figure 5.2 overleaf.

What happens to organic matter in Aerobic Biological Process?
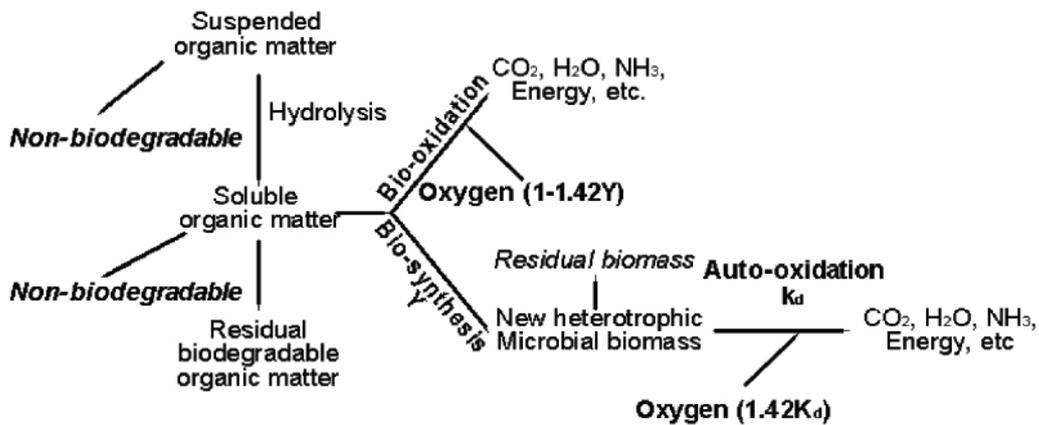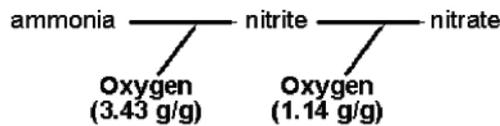
Source: Dr. Akepati S. Reddy, Thapar Centre for Industrial Research & Development, Punjab

Figure 5.1 Aerobic metabolism

What happens to organic matter in Anaerobic Biological Process?
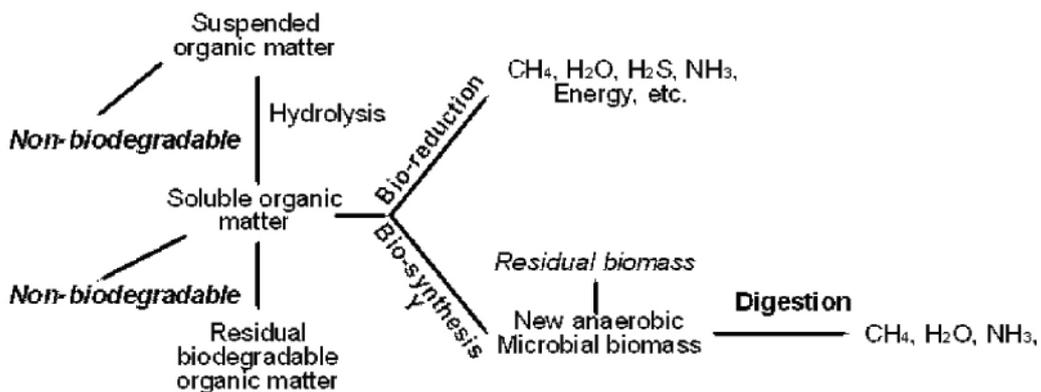
Source: Dr. Akepati S. Reddy, Thapar Centre for Industrial Research & Development, Punjab

Figure 5.2 Anaerobic metabolism

The settled and separated organisms are again put through aerobic or anaerobic processes where their own protoplasm is the reserve food and is referred to as aerobic digestion or anaerobic digestion. The anaerobic digestion is preferred as it yields valuable methane gas, as a source of thermal energy to generate electricity. The digested remains are referred to as digested sludge and can be disposed off as soil filler.

There is also another process of treatment known as facultative where both the aerobic and anaerobic processes occur simultaneously. This is confined to stabilization ponds where the upper portion is aerobic and the settled sludge undergoes anaerobic process at the pond bottom as in Figure 5.3.
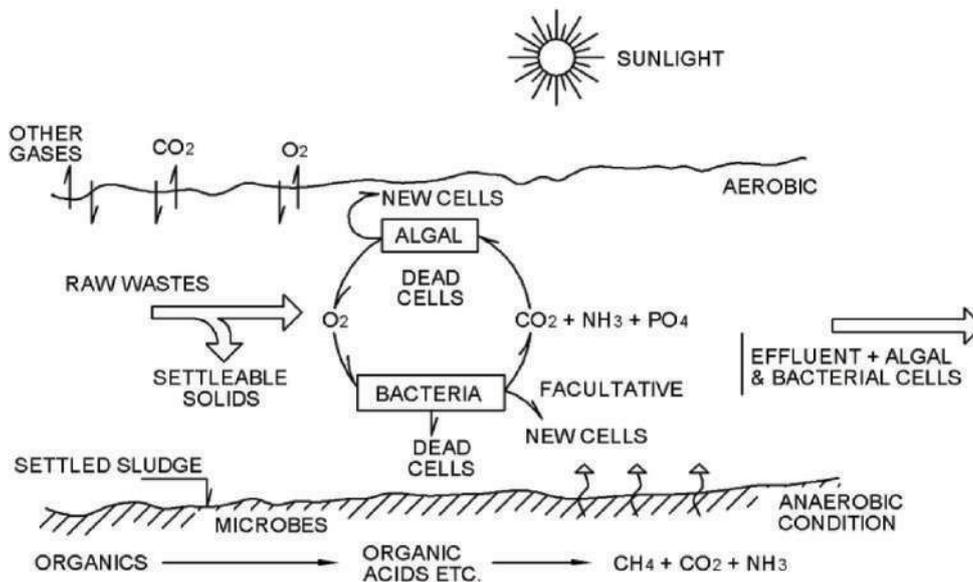
Source: CPHEEO, 1993

Figure 5.3 Facultative Metabolism

The digested organisms are removed once in many years when it reaches about 30 % of the liquid height of the pond and are disposed of as soil filler in the dry summer months.

There are also shallow ponds which are fully aerobic and deep ponds which are fully anaerobic.

5.1.3 Design Sewage Flow

Once the DPR is approved, it takes about three to four years to complete the construction. This completed year is referred to as the base year. Hence, the design population and design volume of sewage shall be taken as the values in the base year.

5.1.4 Raw Sewage Characteristics

5.1.4.1 Determination of Influent Raw Sewage Quality

The value of BOD may vary from place to place due to various prevailing socio-economic conditions, etc. The values may be ascertained for the specific situation with suitable documented justification with laboratory analysis data based on the following procedure.

The raw sewage characteristics are a function of level of water supply and per capita pollution load. Thus, the level of water supply plays a major role in deciding the concentration of pollutants. Other significant factors are settlement and decomposition in sewers under warm weather conditions, partially decomposed sewage from septic tanks, lifestyle of the population, etc. The best way to ascertain the sewage characteristics is to conduct the composite sampling once a week for diurnal variation on hourly basis from the nearby existing sewage outfall or drain.

Considering a four-week month, three samples are to be taken on weekdays, whereas the fourth sample is to be taken on an off day i.e. Sunday.

Sampling for water quality should be conducted for at least one month during dry weather to assess pollution load quantitatively and qualitatively.

The samples should be analyzed for the following parameters;

pH, Temperature, Colour, Odour, Alkalinity, TSS, Volatile SS, BOD (Total & Filtered), COD (Total and Filtered), Nitrogen (NH3, TKN, NO3), Phosphorus (Ortho-P & T-P), Total Coliforms and Faecal Coliforms, TDS, Chloride, Sulphates, Heavy Metals (if there is a chance of industrial contamination)

The results arising from these analyses shall be adopted with the approval of the competent authority. In the absence of drain or outfall, the Table 5.4 can be referred for new developments for 135 L/cap /day rate of water supply. Depending on the rate of water supply the concentrations can be forecast. Based on the raw sewage quality monitoring experiences, the following typical concentrations can be taken for design purpose for 135 L/Cap /day water supply.

Table 5.4 Concentration of various parameters in the absence of drain or outfall

Item	Per capita contribution (g / c /d)	water supply (L / c / d)	Sewage Generation 80 % of (3)	Concentration (mg/L)
(1)	(2)	(3)	(4)	(5)
BOD	27.0	135	108	250.0
COD	45.9	135	108	425.0
TSS	40.5	135	108	375.0
VSS	28.4	135	108	262.5
Total Nitrogen	5.4	135	108	50.0
Organic Nitrogen	1.4	135	108	12.5
Ammonia Nitrogen	3.5	135	108	32.5
Nitrate Nitrogen	0.5	135	108	5.0
Total Phosphorus	0.8	135	108	7.1
Ortho Phosphorous	0.5	135	108	5.0

Illustration BOD = 27 *1000 (mg) / 135 X 0.8 (litres) = 250 mg/L

The Table 5.1 in the manual is retained as a historical value and the Table 5-4 will be followed for the design of biological STPs.

The main reasons for condensing the parameters in the Table 5-4 are

(a) there is a need for listing briefly the parameters of direct relevance to biological treatment processes for BOD removal.

(b) the parameters like total organic carbon, grit, grease, alkalinity chlorides, nitrite, nitrate, potassium are not influencing the biological treatment processes for BOD removal and

(c) the fact that the reduction or elimination of the organisms like coliforms, streptococci, salmonella, protozoa, helminths and virus in these biological treatment processes are incidental and are not specifically designed for.

In general, the required ratio of BOD:N:P is as follows
For aerobic process, BOD:N:P is 100:5:1
For anaerobic process, BOD:N:P is 300:5:1
If the N and P is less, these are artificially added by making a solution of appropriate fertilizers and adding to the raw sewage.

5.2   UNIT OPERATIONS IN BIOLOGICAL TREATMENT

The treatment processes are already explained in Section 5.1. The physical activities used to implement the processes are called unit operations. For example, the physical processes of screening, grit (sand) and suspended solids being settled out are together referred to as primary treatment. The metabolic process is called secondary treatment. Unit operation means the physical activity. For example, simple settling of raw sewage is carried out in primary clarifiers. Pumping air into the sewage for supplying oxygen to the aerobic metabolism is called aeration. Settling of the microorganisms after aeration is carried out in secondary clarifiers. The concentration of settled out organics and microorganisms from primary settling or secondary settling or both together is carried out in sludge thickeners. The anaerobic metabolism of thickened sludge is carried out in sludge digesters. The general sequence is shown in Figure 5.4 and Figure 5.5 overleaf.

5.3   SECONDARY BIOLOGICAL TREATMENT PROCESS

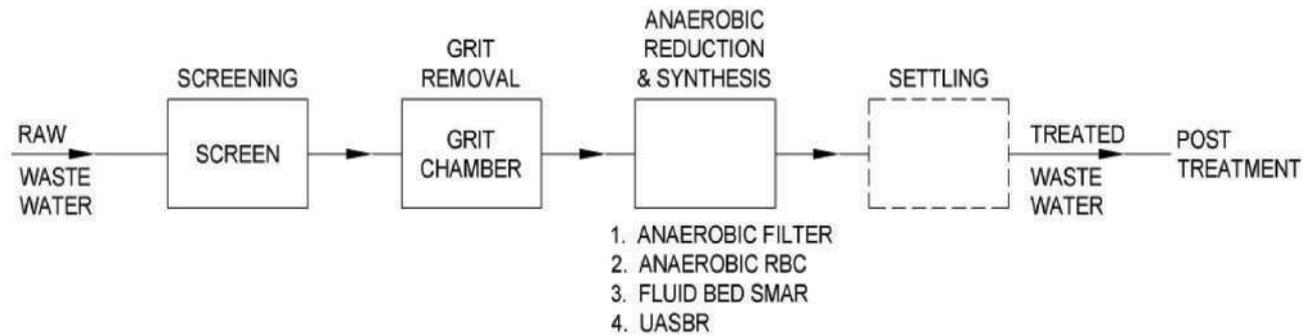### 5.3.1   Aerobic Treatment Process

The following treatment processes as also cited in the advisory issued by the Ministry of Urban Development in March-2012 titled "Recent Trends in Technologies in Sewage Treatment" fall under the classification of aerobic treatment

  • Activated Sludge Process (ASP)

  • Sequencing Batch Reactor (SBR)

  • Moving Bed Bio Reactor (MBBR) / Fluidized Aerobic Bioreactor (FAB)

  • Membrane Bio Reactor (MBR)
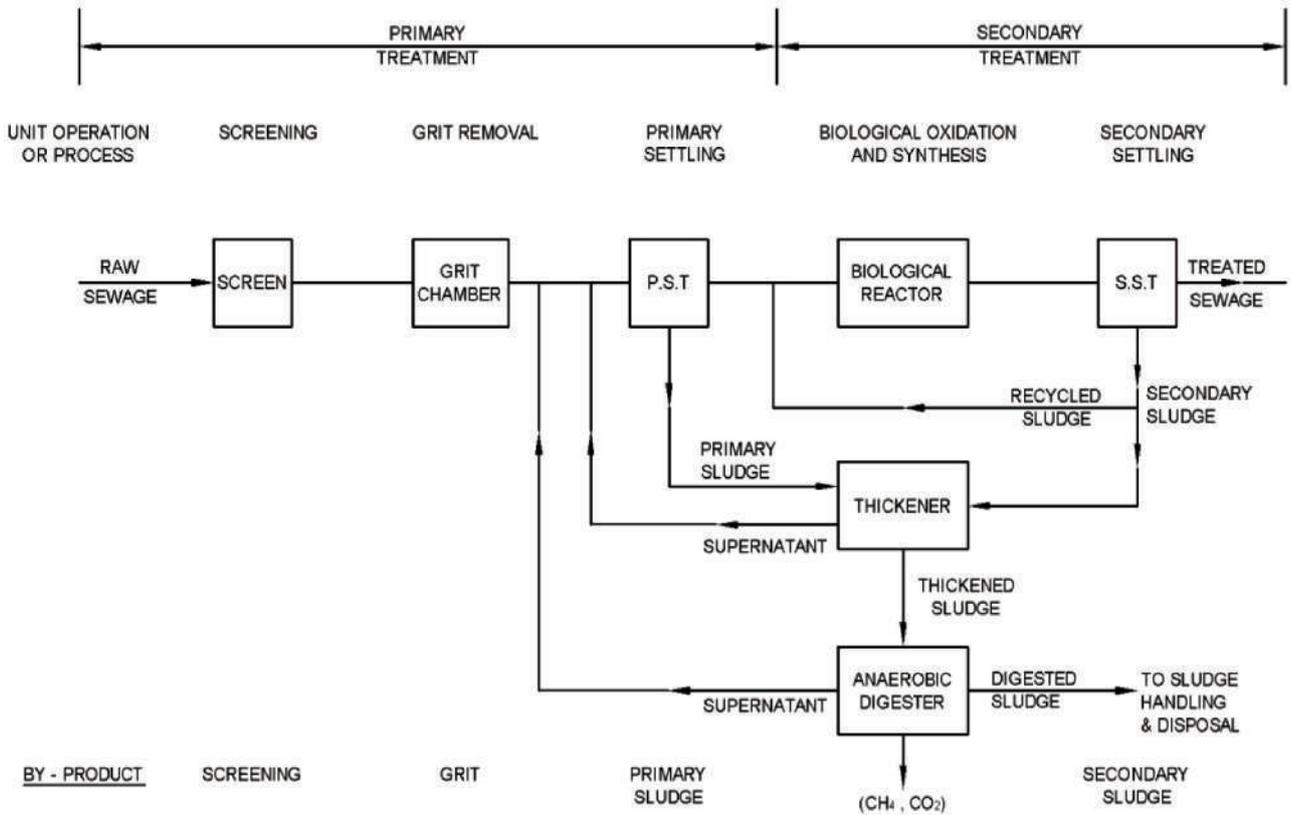
Source: CPHEEO, 1993

Figure 5.4. Unit operations in aerobic mechanized biochemical sewage treatment process
(Secondary Treatment can also be extended aeration and without digester)

Source: CPHEEO, 1993

Figure 5.5 Process flow sheet of conventional anaerobic sewage treatment

- BIOFOR – Biological Filtration and Oxygenated Reactor (BIOFOR)
- High Rate Activated Sludge BIOFOR-F Technology
- Submerged Aeration Fixed Film (SAFF) Technology
- Fixed Bed Biofilm Activated Sludge Process (FBAS)
- Fixed media like Rotating Biological Contactor (RBC)
- Oxidation ditch (O D)

5.3.1.1   Activated Sludge Process

There are two variations of this process namely,

(a) the conventional process for removal of BOD and SS alone and

(b) additionally incorporation of biological nitrification & denitrification for removal of nitrogen in the same process.

Within the conventional process, there are other variations as in Figure 5.6. In the case of very small STPs bleeding excess sludge will be a hydraulic challenge and hence mixed liquor can be wasted intermittently.

The conventional system represents the early development of the ASP which is more than 100 years old. (See Box No. 5.1 page 5-219, about Edward Ardern & William T. Lockett, the inventors)

Over the years, several modifications to the conventional system have been developed to meet specific treatment objectives. In step aeration, settled sewage is introduced at several points along the tank length which produces a uniform oxygen demand throughout.

In tapered aeration, air supply is tapered to match the needs from the deeding point of sewage to its exit from the aeration tank.

Contact stabilization provides for reaeration of return activated sludge from the final clarifier, which allows a smaller aeration or contact tank.

While conventional system maintains a plug flow hydraulic regime, completely mixed process aims at instantaneous mixing of the influent waste and return sludge with the-entire contents of the aeration tank.

The extended aeration process employs low organic loading, long aeration time, high mixed liquor suspended solids (MLSS) concentration and low F/M. Because of long detention in the aeration tank / oxidation ditch, the MLSS undergo considerable endogenous respiration and get well stabilized and in these cases, the excess sludge does not require separate digestion and it can be directly dried on sand beds or dewatered in equipments. In addition, the excess sludge production is minimum in this case. The conventional system, the complete mix and the extended aeration have found wider acceptance.

5.3.1.2   Fixed Media System

The primary sedimentation is a pre-requirement in these applications. These are the older trickling filters with stone media and now use synthetic media such as inclined corrugated media placed in cube sized packs and the inclinations changed to opposite directions in successive layers.

The applied sewage is distributed from the top of the media pack by a stationary or hydraulically driven reverse jet arms on opposite radii or rotated by a mechanical drive. This arrangement is needed to apply the sewage on the entire plan area uniformly. This sets up a hydraulic draft and allows the lighter gases of metabolism to escape upwards and fresh air to rush in at the bottom through open ports on the side walls at the floor.
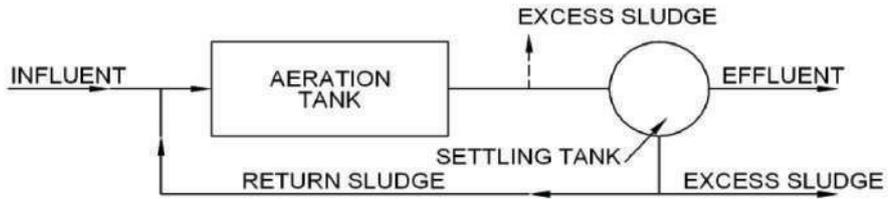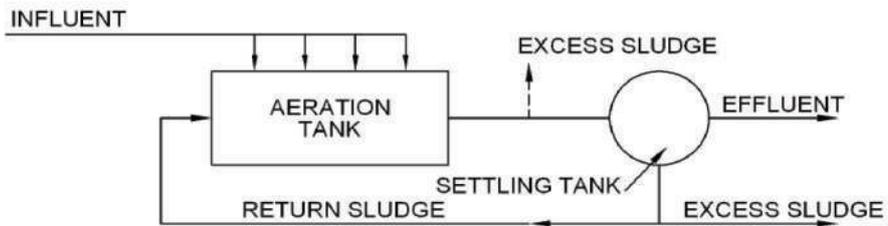
FIG.(a): CONVENTIONAL ACTIVATED SLUDGE
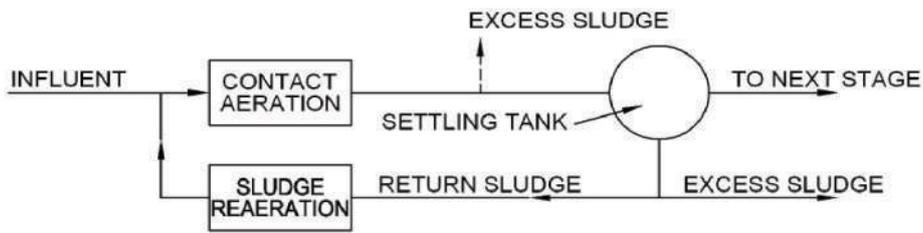
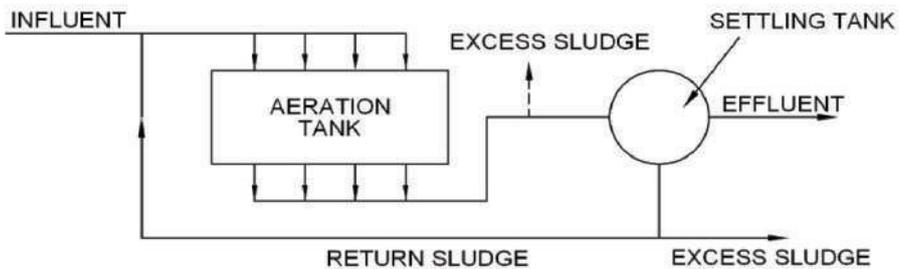FIG.(b): STEP AERATION

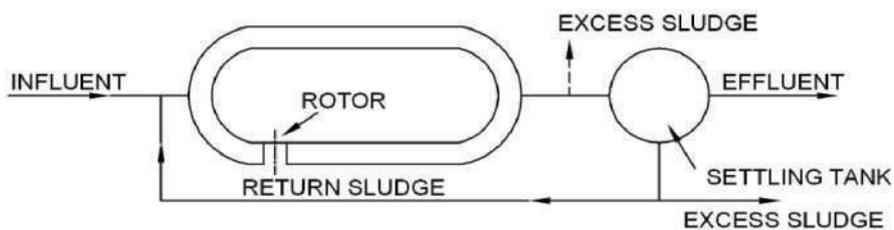FIG.(c): CONTACT STABILIZATION

FIG.(d): COMPLETE MIX PLANT

FIG.(e): OXIDATION DITCH

Source: CPHEEO, 1993

Figure 5.6 Schematic diagrams of activated sludge treatment with different modifications

The organisms grow as a film on the fixed media and bring about the metabolism as the sewage passes over them as a film. In due course of time, the thickness of the film increases. This results in the film shearing away from the media which is called sloughing. This is carried away to secondary settling tanks. Recirculation of the settled sewage is sometimes practiced to the inlet of the reactor. This helps to return the enzymes released by the microbes back to the reactor for solubilizing the sewage organic matter. The media should be only made of virgin material like HDPE, PVC. The fixed film media system is in Figure 5.7.
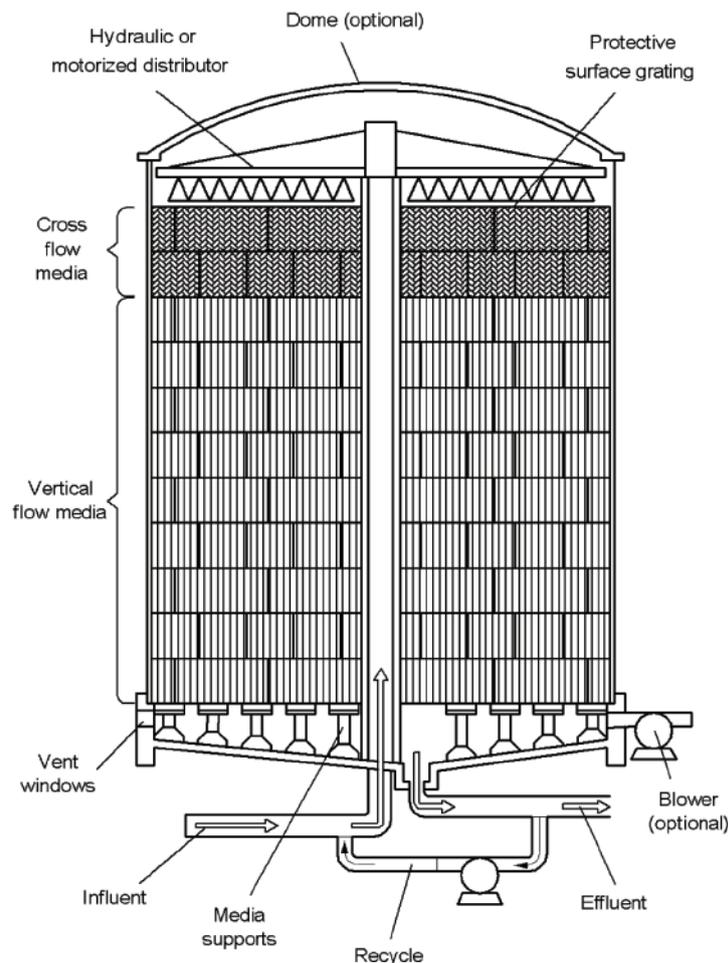
Figure 5.7 Fixed film synthetic media filters

5.3.1.3   Moving Media Systems

These involve the synthetic small sized media, which are fluidized in the reactor by artificial air supply by compressed air released at the floor of the reactor. This brings about the circulatory movement of these media into the tank contents. The trade names are Fluidized Aerobic reactor (FAB), Moving Bed Biological reactor (MBBR) as also Fluidized Anaerobic reactor and their operational principle is illustrated in Figure 5.8.

The microbial film that develops over the surface of the fluidized media permits the metabolism. Secondary settling is needed in the case of FAB and MBBR. In the case of FAB, additional further treatment may also be necessary.
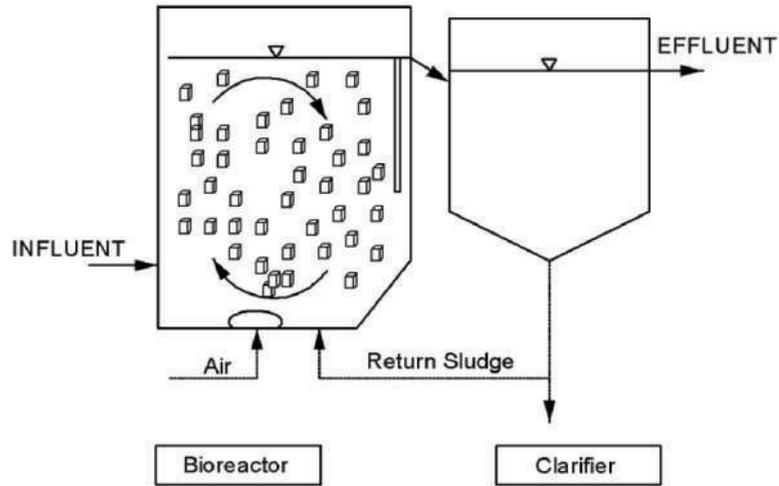
Figure 5.8 Fluidized aerobic bed reactor showing the media in motion

5.3.2   Anaerobic Treatment Systems

The following treatment processes fall under the classification of anaerobic treatment.

a) Up flow Anaerobic Sludge Blanket - UASB

b) Anaerobic filter - AF

c) Anaerobic fluidized bed

These are mainly needed in case where bio-methanation is possible to recover the energy. Instances are the Up Flow Anaerobic Reactor and sludge digesters and the schematic of these are shown in Figure 5.9, Figure 5.10, and Figure 5.11. The principle of anaerobic treatment is already shown in Figure 5.2.
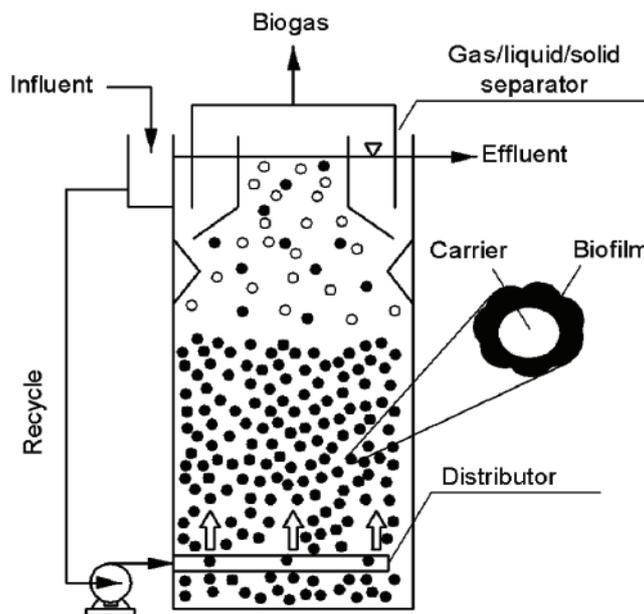
Figure 5.9 Fluidized anaerobic bed reactor showing the media in motion
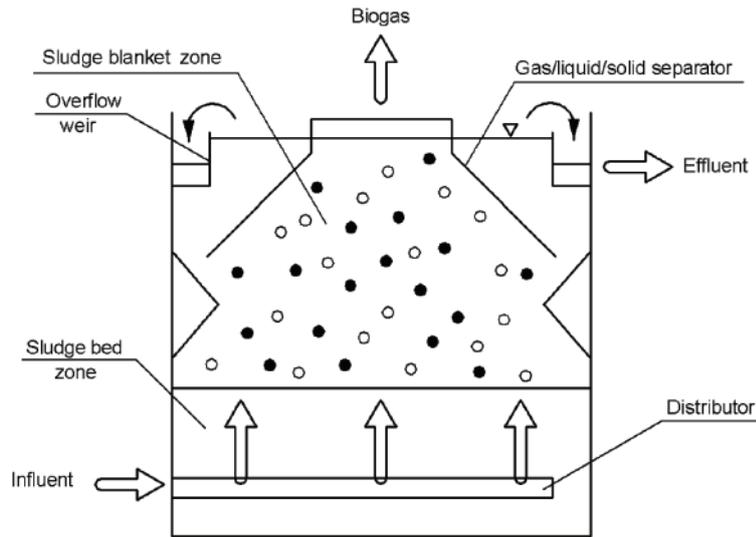
Figure 5.10 Up flow anaerobic sludge blanket reactor
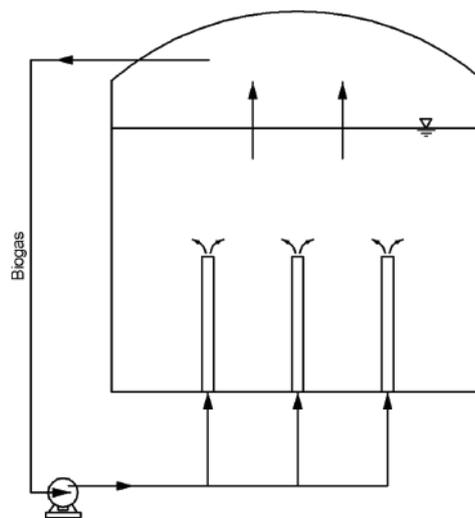
Figure 5.11 Anaerobic sludge digester mixing with generated biogas

5.3.3   Facultative Treatment Processes

The following treatment processes fall under the classification of facultative treatment.

a) Aerated lagoon - AL

b) Waste stabilization pond - WSP

c) Eco Bio Block - EBB

5.3.4   Performance Efficiency of Conventional Treatment Processes

The performance efficiency of the conventional treatment processes are in Table 5.5.

Table 5.5 General Treatment Efficiencies of Conventional Treatment Processes

Process		Percentage Reduction		
		SS	BOD	Total Coliform
1	Primary treatment (sedimentation)	45-60	30-45	40-60
2	Secondary treatment			
	(i) Activated sludge plants	85-90	85-95	90-96
	(ii) Stabilisation ponds (single cell)	80-90	90-95	90-95
	(iii) Stabilization ponds (two cells)	90-95	95-97	95-98

Source: CPHEEO, 1993

5.3.5 STP Land Area

In recent times, population densities have increased in ULBs. Even public lands are being encroached upon and these become difficult to vacate. Thus getting open vast land areas for setting up STPs is a problem.

Moreover, because of densification, even if open lands are available, the population is very close to these lands and they object to the STPs nearby. Thus, the STPs have to be planned to occupy lesser areas than the STPs of olden days.

This has brought in the concept of multi-tier STPs where the primary clarifier can be at the topmost floor with aeration tank below it and secondary clarifier at the bottom floor. It is also possible to construct multi-tier STPs like the SBR based STP in Bangkok as in Figure 5.81 (later on in this chapter) whereby land area can be reduced. Similarly, MBR also results in lower area because of higher MLSS concentration and reduced volume of aeration tanks permitting vertical arrangement of tanks one over the other.

An important engineering requirement in such cases is the need to ensure headroom of 4.5 m in between the top of sidewall of the bottom tank and the roof of the upper tank. This is because of the fact that these locations come under industry classification and electrical utilities are involved.

The recurring cost of additional pumping of all the raw sewage over the entire increased height has to be considered in such cases.

There are also similar STPs in Japan (Figure 5.12 and Figure 5.13) which have been built conserving space. These types of facilities have a complicated structure compared to the conventional facilities. Therefore, while making decision on adopting these facilities, the difficulty in O&M of these facilities has also to be well considered.
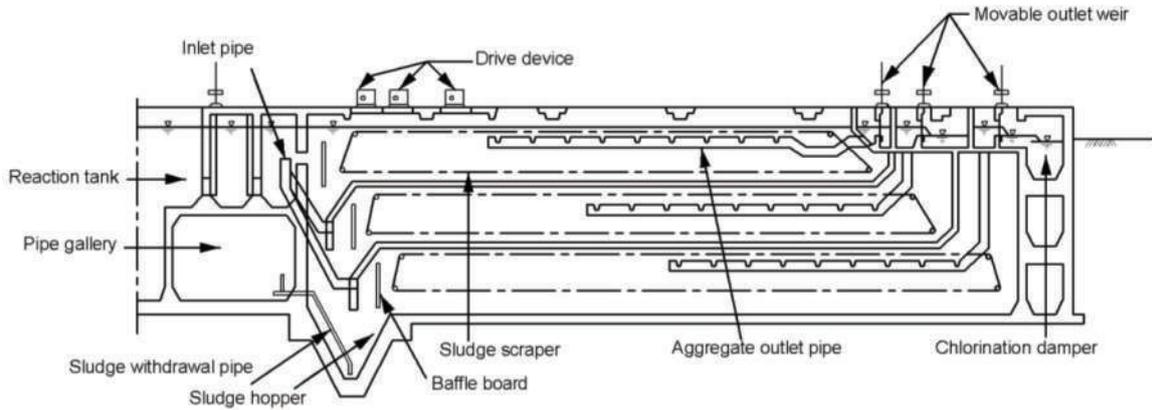
Source:JSWA, 2009

Figure 5.12 Example of secondary clarifier built in 3 layers (Hirano STP, Osaka, Japan)
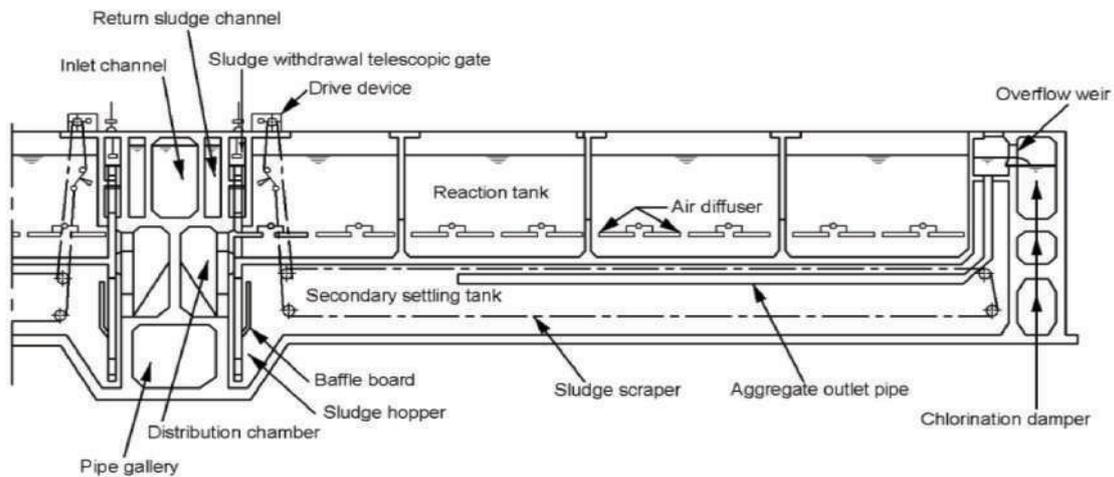
Source: JSWA, 2009

Figure 5.13 Example of reaction tank and secondary clarifier in layers (Imafuku, Osaka, Japan)

5.3.5.1   STP Land Area for different technologies

The STP land area required for various treatment technologies are shown in Figure 5.14. It can be observed that excluding WSP the land area is in the range of 0.2 to 1.0 hectare per MLD for STP as per the technology adopted keeping in view the size of the town / area.

The minimum foot print will also be an important factor in evaluating the treatment technology. This is because in the case of Koyambedu 125 MLD STP at Chennai for the CMWSSB, the cost of the land based on official rates exceeded the cost of the STP itself. This was because the location of the STP site is in the prime hub of metropolitan transport, metro rail, wholesale vegetable market, long distance bus station, etc. This however may not be the case everywhere. Hence, the decision on relative costing of savings in land area is best left to the concerned ULB's depending on the nature of the site and its potential uses.
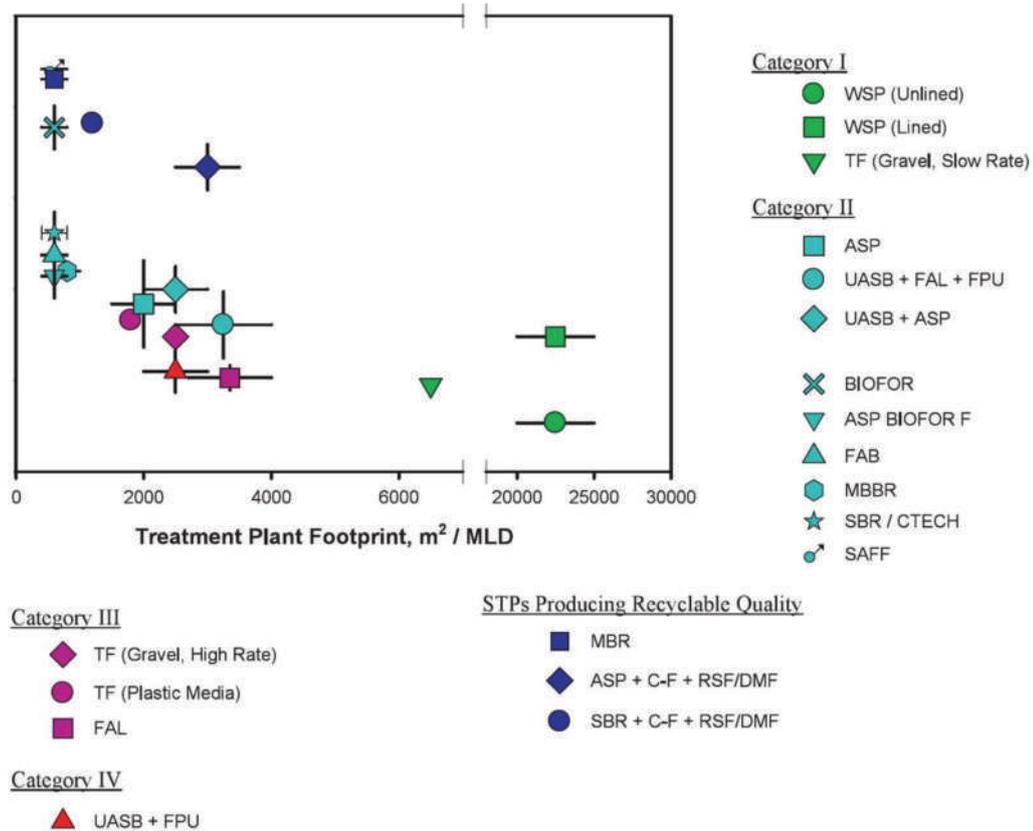
Source: NRCD, MoEF, 2009

Figure 5.14 STP Land Area required for various treatment technologies

Considering the foregoing, the energy cost, operating cost and capital cost will be the determining factors in DPR's while looking into the technologies. The land cost is to be kept out at the DPR stage.

5.3.6 Arrangement of Treatment Units in STP

The preferred arrangement will have the following guidelines

• The civil construction of units can be integrated by common walls especially sewage holding structures

• This can be either rectangular or square tanks touching each other or circular tanks inscribed concentrically.

• This will economize on costs of piping because sewage can be conveyed by channels through all such units.

• Stop gates operated by hand wheels on a rack and pinion method can replace costly buried valves and avoid the difficulty of O&M of such buried valves.

• An illustration of one such construction is the biological STP at the Indira Gandhi International Airport terminal T3 as illustrated in Figure 5.15.

Source: Indira Gandhi International Aiport, Delhi

Figure 5.15 An illustration of a nitrification-denitrification bioreactor in India

- The anoxic tank, aeration tank and clarifier are all arranged in a single civil structure for the 17.5 MLD STP designed with two parallel modules. Photographic views in construction and in O&M are shown in Figure 5.16
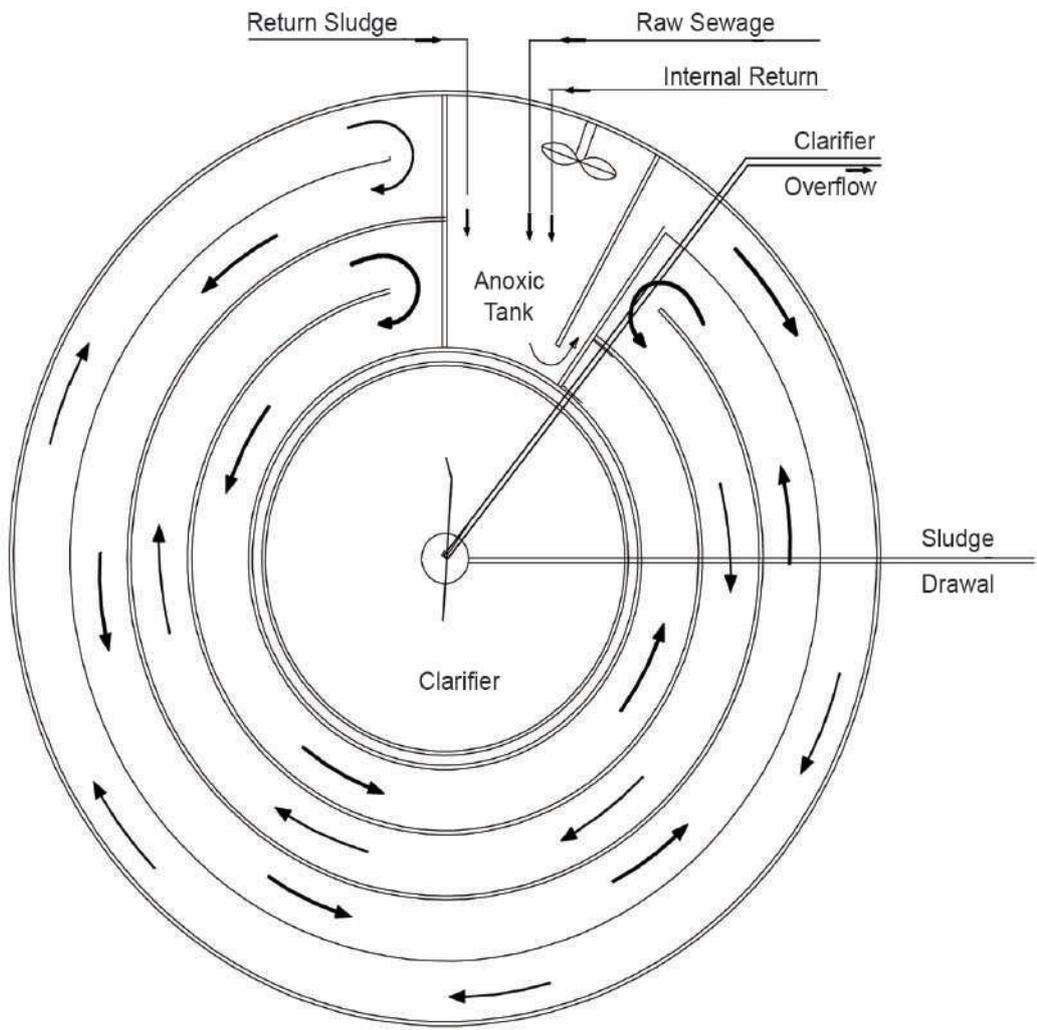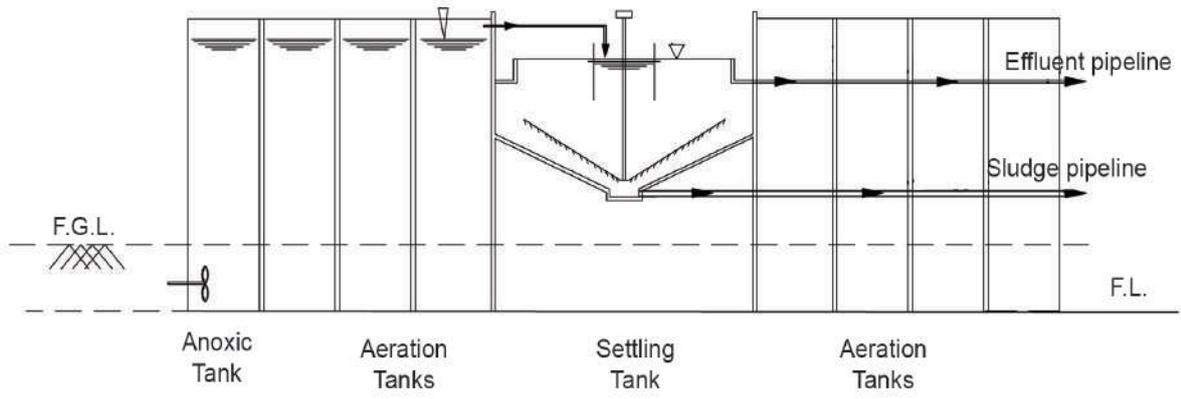
Source:Indira Gandhi International Airport, Delhi

Figure 5.16 Left is the bioreactor during construction; Right is the bioreactor after commissioning showing the rich MLSS culture in the annular plug flow reactor

5.3.7   Control of Hydrogen Sulphide Odour

Whichever be the sewage treatment process that is used, care should be taken to avoid unnecessary stagnation of raw sewage or sludge. At the same time, there are the following locations from which the odour of Hydrogen Sulphide can arise.

a) Sewers that are choked and not flowing,

b) Sewage pumping station sumps where sewage is not pumped out then and there,

c) Primary clarifiers, sludge thickeners, digesters and sludge drying beds in STPs.

The standardizing of technologies for odour control in such locations is yet to be validated. This is due to the factual position that until now, the SPS and STP are located fairly away from the habitations. However, of late with increasing urbanization and densification of land use, the locations of these are almost coming up in the midst of urban living. Hence, it becomes necessary to institute odour control measures in sewerage and sewage treatment systems.

In sewers, keeping the sewage elevation below the crown of the gravity sewers will enable outside air to be drawn and it will rise through the sewers at the rising gradient and thus avoid build up of the foul odours.

In SPS, covering the raw sewage sump is not advised because the submersible pump sets have to raised and lowered for maintenance. The suction of the sewage by the pumps enables the inward flow of the air and this helps in avoiding build up of foul odours.

In the case of STPs, the locations are screen chambers, detritors, equalization tanks, primary clarifiers, sludge thickeners, sludge digesters and sludge dewatering units.

The prevailing technology options for odour control in STPs are compiled in Appendix A.5-1. In essence, the options are in providing a cover over such units to contain the odourous gas and air mixture and then draw it into a gas purification unit and then condition the purified air to be pumped back into the enclosure.

The illustrations of the hydrogen sulphide stripping units described in Figure 6.14 in Chapter 6 can be followed. Their design criteria will be as per the respective equipment manufacturers.

For this purpose, the layout of the various unit operations is best arranged in such a way to group the odour causing units together for a single master enveloping dome instead of multiple domes over each of the units. At the same time, the contained space must permit access to the mechanical equipment for repairs, renewals etc. In general, a circular configuration will be relatively advantageous in economically providing a funicular polygon type of dome using a translucent, synthetic non corrosive light weight material for the dome. The dome itself can be raised free of the structure during the repairs and renewals to provide unhindered access and visibility in work. During the containment position, the free air space above the unit can be minimal so that the volume of air-gas mixture to be purified will be minimal and will permit higher concentration of the gas which will help in better efficiency in purification. One such layout at the Thiruvottoyur 31 MLD capacity STP of CMWSSB now under construction in the peripheral urban location of Chennai is shown in Figure 5.17.
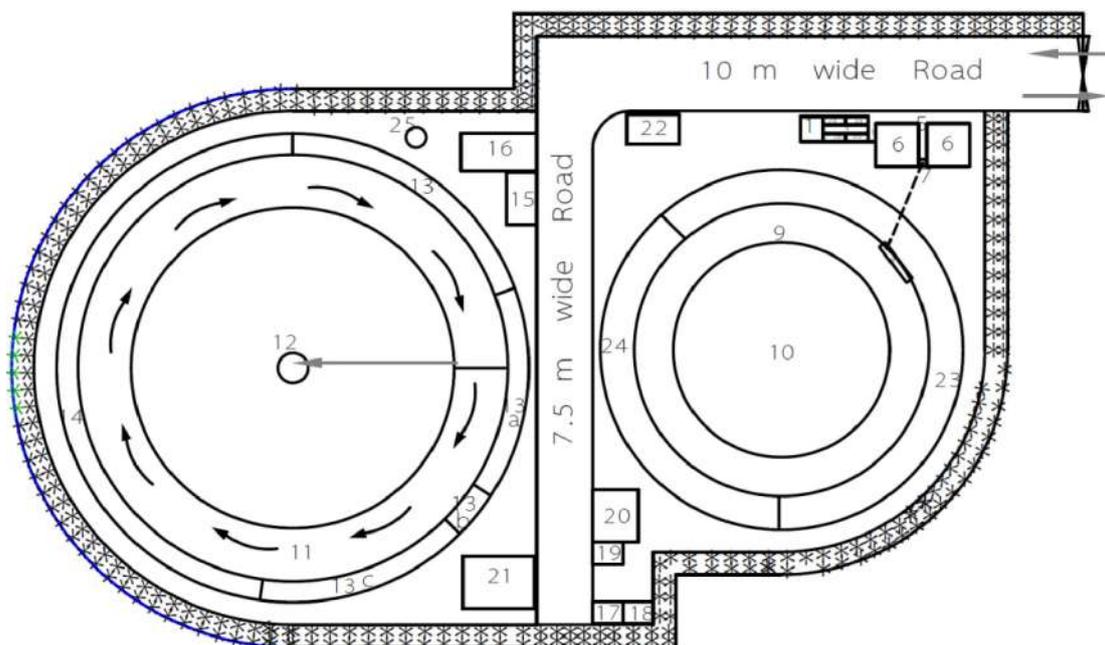
Figure 5.17 Layout of the civil units at the 31 MLD Thiruvottiyur STP of CMWSSB

The circular layout at right incorporates the raw sewage equalization tank (9), primary clarifier (10), sludge thickener (23) and sludge digester (24). The circular unit at left incorporates the aeration tank (11), secondary clarifier (12), tertiary filter (13) sumps for sludge, filtrate etc (13a to 13c) and chlorine contact tank (14). It may be seen that the odour causing units of raw sewage equalization tank, primary clarifier, sludge thickener and sludge digester are laid out into a single integrated structure for easy odour control and additional dome- type enclosure facility in later days.

5.3.7.1 Buffer zone around the STP

Adequate measures may be taken for de-odourization in the STP. The units in the STP which need deodorization system are screen chambers, grit chamber, primary clarifier, sludge thickening and dewatering units. However, the odor emissions are negligible for sludge treatment facilities of extended aeration systems due to in-situ aerobic digestion of sludge. Wherever the STPs are provided with de-odourization system, specific buffer zones are not required.

In case of STP's where de-odorization system cannot be provided, an aerial / peripheral distance of 100 m from the odour-producing units to the habitation is recommended. However, this distance can be reduced by conducting public consultation.

5.3.8 Required Engineering Data

The mandatory data shall include:

a) Contour map and elevations connected to the nearest Survey of India permanent bench mark.

b) Soil tests till it attains hard stratum and test bores at a grid of 500 m squares

c) Rainfall of nearest observatory for at least 50 years

d) Highest intensity of rainfall

e) Earthquake records of the region

f) Maximum flood level at the site and in the identified receiving water

g) Wind rose.

5.3.9 Design Period

This shall be as in Table 2.1of Chapter 2.

5.3.10 Sewers and Conduits

Generally, urban sewerage schemes involve large volumes of sewage. The sewers conveying it to the STPs can be pipes or in-situ constructed conduits passing through dense areas. These conduits shall be preferably non-corrosive materials such as CI or DI pipes.

They can also be RCC pipes with high alumina cement lining and adequate wall thickness to withstand the traffic loads in public roads. Pre-stressed concrete pipes are not recommended.

This is because if the steel is corroded by Hydrogen sulphide from the sewage, it will lose its thickness. This will affect the ability of the steel rods to withstand the pre-stressed tension and they may snap. Then the entire pipe will crack. These cannot be repaired easily at site.

The old brick-arch sewers are good against corrosion, but the challenges are procuring quality bricks, availability of dedicated skilled masons and difficulties of future repairs if needed.

5.3.11   Installation of Mechanical Equipment

The general arrangement (GA) drawing of each mechanical equipment erected in its civil works tank is called the GA drawing. These drawings shall show clearly the way the bolts, nuts etc are securely fixed in the civil works. It is better if the drawing clearly states the sequence of removing the equipment later on, when needed. It shall be the basis for the erection of the equipment. All steel parts shall be sand blasted and then painted with the approved primer and first coat before it can be assembled at site. All bought out equipment parts shall be once again checked at site for scratches and peelings of paints and rectified before permitting the erection. Levelling of overflow weir edges shall be done by guiding the finished levels with the help of two levelling instruments placed perpendicular to each other and focusing on the same reference at a given time. The run of mechanical equipment first involves the run of electrical prime movers and shall not be run for unduly long periods. The sludge scrapper in clarifiers shall be completed in their erection along with screeding of the inclined floor.

5.3.12   Unit Bypasses

Bypass arrangements shall be by gravity either by a direct outfall conduit or through a dedicated terminal tank before the outfall conduit. In the case of a tank, its volume shall be equal to the buffer volume needed as calculated by mass diagram (Appendix A.5.2). However, the economics of such a dedicated terminal tank versus the size of the outfall conduit to handle the peak flow shall also be considered before arriving at a decision. It should be possible to chlorinate this volume to the same level as for treated sewage before discharge. If pipes or conduits of RCC are used, they shall be with protection by High Alumina cement or sulphate resistant cement. They can also be of other pipe materials or brick masonry conduit. All such pipes or conduits shall be designed for a velocity of not less than 0.6 m/s and not more than 2.5 m/sec.

The bypass facility is to be provided after each location as the screen chamber, detritor and the primary clarifier. There shall not be any bypass before the secondary clarifiers of activated sludge plants because the MLSS will be washed out. The chlorinator should be designed to handle a dosage of 10 mg/L. The volume of the chlorine contact tank shall be 30 minutes based on the average flow. The discharge location shall be got approved from the PCB. Its physical arrangements shall be carried out under intimation to the pollution control authorities and the authority in charge of the waterways.

5.3.13   Unit Dewatering

Dewatering of treatment units is not a standard requirement in STPs because the sewage flow is for 24 hours 7 days a week. However, if civil work in a tank is needed for maintenance, the dewatering becomes necessary. All units shall be constructed with a puddle flange pipe of not less than 150 mm ID in the side-wall just above the formation level and provided with an isolation valve and closed by a removable blank flange on the airside. A portable diesel based pump set can be connected at this flange to pump out the contents into the plant bypass chambers. This pumping is necessary because these chambers in the by pass pipe-line will have their sidewalls at the same elevation as the sidewalls of treatment units, and hence, gravity dewatering is not possible.

5.3.14   Floatation

Floatation as a method of removing the suspended matters is not usually followed in biological STPs. Their only application is in removing oil & grease from raw sewage if industrial uncontrolled sewage enters into the sewage in the collection system. The oil & grease enters the sewers as free oil and grease. However due to the turbulence of the flow in sewers, pump sets, pumping mains, etc. It results in water and oil forming an "emulsion". These will not float like oil & grease and they remain distributed in the entire sewage. Thus they cannot be removed directly by floatation. The emulsions in sewage are usually oil in water emulsions. These emulsions must be first broken up by chemical reactions to release the free oil and grease. Then only they can be removed by floatation. This breaking of the emulsions is done by adding chemicals like alum or polyelectrolytes. This is referred to as "de-emulsification". Thereafter the sewage can be allowed to stand still to allow the de-emulsified oil and grease to float. This is removed by skimming and is called scum. If fine air bubbles are intimately dispersed into the de-emulsified sewage by passing the mixture through pressurised tubes, the bubbles get more intimately bonded to the oil and grease particles. Thereafter the mixture is released immediately into a shallow tank. Here the air bubbles and the de-emulsified oil and grease rise rapidly as scum, and are removed by skimmers. The subnatant of the tank is the oil & grease removed sewage. These are all patented units and overall design criteria cannot be prescribed. Usually this is provided after the screens and grit removal.

5.3.15   Construction Materials

5.3.15.1   Concrete

This section gives only the guidelines and not exhaustive specifications. In general, RCC is the preferred material for civil structures and shall follow IS: 456 and IS: 3370. The cement shall be of IS: 269 with the latest revision thereof for 33 grade ordinary Portland cement, IS: 12330 for sulphate resistance cement, IS: 8112 with the latest revision thereof for 43 grade ordinary Portland cement and IS: 12269 with the latest revision thereof for 53 grade ordinary Portland cement as the case may be. Sand shall follow IS: 2116. Coarse and fine aggregates (stone and sand) shall follow IS: 383 and IS: 456. The maximum quantities of deleterious materials in the aggregates, as determined in accordance with IS: 2386 (Part II) shall not exceed the limits given in Table I of IS: 383 All Reinforcement steel should be either of the following:

(a) Fusion Bonded Epoxy coated having not less than 175 microns thickness and up to 300 microns to reinforcement of all diameters as per IS: 13620 for RTS rods. (b)The binding wire should be of PVC coated and the exposed portion if any after bar bending work should be covered with the Epoxy paint supplied by the coating agency. (c) CTD of high strength deformed CRS steel reinforcement bars conforming to relevant BIS codes (Gr Fe 415, BIS code 1786-1985) from producers approved by the Ministry of Steel (MoS) of GOI.

Water proofing on the inside surface of all liquid retaining structures (except sludge digester and gas holding tank, which shall be of sulphate resistant cement and solvent free epoxy painted) shall be made by thoroughly cleaning of all the dust, grit, grease, oily matter, and other deleterious material.

This will be followed by three coats of sodium silicate solution in a proportion of 1 part of sodium silicate to 3 part of water applied for one litre of solution, to cover 4 sqm of surface, and each coating allowed to dry for 24 hours.

5.3.15.2   Structural Steel

Wherever a structural steel member like channels, angles, I sections etc used in mechanical equipment, gets in contact with sewage it requires special precautions. The sludge scrapers in the floor of clarifiers are held in position by mild steel angles suspended from the walkway platform of the clarifier. These are partly inside the sewage and partly above it. The wind action of the sewage causes oscillation of the sewage surface. This results in alternate exposure of steel to wetting and drying. This condition accelerates the corrosion. In such cases, the steel member shall be spliced with SS members for 30 cm above and 30 cm below the sewage surface. Epoxy or special polymer painting of the other portions of steel is needed. Before painting, the bare steel shall be sand blasted to SA 2.5 Swedish standard SIS 05 5900 with a surface profile not exceeding 65 microns or the equivalent specifications of ASTM. Where sand blasting is not possible, manual chipping or wire brushing to remove loose rust and scale shall be permitted to ST 2 Swedish standard SIS 05 5900. Solvent free epoxy coating shall be for 360 microns and curing shall be done for 7 days, at room temperature if the temperature is less than 15ºC, the surface shall be warmed up by in candescent lamps, heaters, blowers or infrared lamp.

5.3.15.3   Steel Pipes

Bar / wire wrapped steel cylinder pipes with cement mortar lining and coating (including specials) conforming to IS: 15155 shall be permitted for large diameter pipelines that are not laid under concrete structures but only under roadways and freeways. For pipelines under concrete structures, preferred material shall be DI, CI or high alumina RCC pipes and with O-ring joints for all these.

5.3.16   Coating and Painting

Already covered under Section 5.3.15.1

5.3.17   Operating the Equipment

All the operations of equipment shall be carried out as per the preventive maintenance manual of the respective equipment manufacturer and an independent quarterly external third party inspection and certification.

5.3.18   Erosion Control during Construction

The site shall be first set out with a ditch drain in kutcha earthwork with stone boulders loosely placed in a trapezoidal section to intercept all overland runoffs from the surrounding areas onto the STP site. The termination of the ditch drain shall be into a public water course by gravity and if this is not possible a dyke and temporary diesel driven pump sets shall be readied before the monsoon and manned 24 hours seven days a week for the full monsoon period.

Where this type of prevention appears too complicated for reasons like land availability etc., a diaphragm wall shall be first constructed all around the STP site with its sill at least 50 cm above the MFL of the area at the entry and exit at the STP site. Within the STP site, temporary ditch drains along the proposed road alignments shall be provided and the storm water pumped out to the nearest water course. The use of well point system is also recommended.

5.3.19   Grading and Landscaping

The grading of the finished site shall be such that the riding surface of the roads shall be at least 20 cm above the finished ground level on both sides. There shall be a suitable chamber to drain the storm water to the drains on both sides. In main arterial roads, the free land between the edge of storm water drains and the nearest structure shall be not less than 3 m to permit the laying and maintenance of water pipelines, sewers, manholes, power cables, street-lights, instrumentation cables and interconnecting pipelines between STP units. In advanced countries, the cables and interconnecting pipelines between units are taken through RCC walk through box culverts connecting the units below GL. This permits their future maintenance by walking through the box culverts and without any digging up the formed ground. Such man entry box culverts shall follow all the safety precautions for confined spaces and especially the indoor air quality by forced ventilation and adequate lighting and emergency communication facilities. Landscaping shall be confined only to turfing. Flowering plants if used shall be housed only in dedicated ornamental pots or concrete troughs. Trees with spread out roots should never be permitted within the STP site and for clear 6 m from any civil structure. This is because, these roots are known to go in search of water and even pierce through the sidewalls and floor of concrete structures. Thereafter, they will corrode the reinforcement steel rods and weaken the concrete side walls. A good grading and landscaping is shown in Figure 5.18, Figure 5.19 and Figure 5.20 at the Vrishabhavathi Valley STP at Bangalore. The fountain in Figure 5.19 is with the treated sewage.

Source: BWSSB

Figure 5.18 Grading and landscaping at Vrishabhavathi Valley STP, Bangalore

Source:BWSSB

Figure 5.19 Treated Sewage Fountain at Vrishabhavathi Valley STP, Bangalore

Source: BWSSB

Figure 5.20 Grading and landscaping at Vrishabhavathi Valley STP, Bangalore

5.4   PLANT OUTFALLS

The outfall takes the treated sewage from the STP to the disposal location. It can be either a gravity conduit or pumped conduit. The guidelines of section 5.3.10 apply here also. It shall be designed for the DWF if equalization basin is provided in the STP. If not, it shall be designed for the peak flow.

It shall be constructed with the invert at minimum of 0.5 m above the MFL of the receiving water body or other regulations by the local authority. The discharge shall be first let into a receiving well and then spill over to the river or water body. The designs of the receiving well shall be got approved by the competent local body before construction.

The receiving well shall be provided with RCC removable cover slabs of heavy duty such that where required, the full plan view of the well can be ventilated to permit man entry safely when required. The top of the cover shall be sloping with a finish of coarse aggregate like a stucco finish to ensure that it is not used for squatting or as a leisure spot.

There shall be two heavy duty manhole covers on the slab at opposite ends and a stub ventilating pipe with a downward "Tee" and 90 degree bends at each end duly covered with synthetic mosquito mesh secured by nylon ropes with their ends heat sealed.

5.4.1   Discharge –Physical Impact Control

The main objective is to avoid erosion of the banks of the water course or river or reservoir due to the hydraulic turbulence of the treated sewage falling and running over these earth surfaces. This will be controlled by the spillway design.

5.4.2   Protection and Maintenance of Outfalls

This shall be done only by the competent local authority in charge of the water course or river or reservoir. The annual cost shall be paid to them as part of O&M cost of the STP.

5.4.3   Sampling Provisions

Sampling provisions are needed for grab sampling in case of manual collection and automatic instrumentation based on-line testing for automated sampling. The sample collection facility shall be provided in the firm and level ground portion before the bunds and on the roadside.

This shall be by means of a scour pipe enclosed in a masonry chamber with removable heavy duty CI manhole framelid. Trying to sample at the exact discharge end is not necessary as it involves risk of the person falling into the water course or river or reservoir.

5.5   ESSENTIAL FACILITIES

5.5.1   Emergency Power Facilities

The emergency power facilities will be as in Section 5.12 in the electrical section.

5.5.2   Water Supply

The water supply at the STP shall be provided on the same lines as a public water supply system as in the CPHEEO manual on water supply and treatment.

5.5.3 Sanitary Facilities

Toilets and baths in the STP site are to be judiciously located so that the operator need not have to walk long distances. As the sewage flows at these locations will be very small and intermittent, septic tanks shall be provided at each location. The tank shall be emptied by a sewer lorry as part of the O&M segment of the city sewerage system. It shall discharge it into the raw sewage receiving structure of the STP.

5.5.4 Flow Measurement

This shall be as per Section 4.15.

5.5.5 Laboratory

This is dealt with in Appendix A.5.3, Appendix A.5.4 and Appendix A.5.5.

5.6 SCREENING, GRIT REMOVAL AND FLOW EQUALIZATION

5.6.1 Screening

Screening is essential for removal of floating materials which are mainly sachets, plastic sheet bits, leaves, fibres, rags, etc. If these are not removed, they will get into the pumps and entangle in the impellers. They can also be drawn into suction pipes and choke them and it is difficult to locate their position in the pipeline. They can cause objectionable shoreline conditions where disposal into sea is practiced. Screens are used ahead of pumping stations, meters and as a first step in all STPs. A screen is a device with openings generally of uniform size. The screening element may consist of parallel bars, rods, gratings or wire mesh or perforated plates and the openings may be of any shape, although generally they are contrived from circular or rectangular bars. It is recommended that three sequential stages of screens shall be provided being coarse, followed by medium and followed by fine screens. A typical design example is shown in Appendix A.5.6.

5.6.1.1 Coarse Screens

They serve more as protective devices in contrast to fine screens, which function as treatment devices. Coarse screens are usually bar screens and are sometimes used in conjunction with comminuting devices. A bar screen is composed of vertical or inclined bars spaced at equal intervals across a channel through which sewage flows. It is usual to provide a bar screen with relatively large openings of 25 mm. Bar screens are usually raked clean manually or by mechanical devices. These rakes sweep the entire screen removing the floating substances. Some mechanical cleaners utilize endless chains or cables to move the rake teeth through the screen openings. Screenings are raked to a platform with perforations which permits the drainage of water content back to the unit.

Hand cleaned racks are set usually at an angle of 45 to 60 degrees to the horizontal to increase the effective cleaning surface and facilitate the raking operations. Experience indicates that the area of the vertical projections of the space between the bars measured across the direction of the flow should be about twice the area of the feeding sewer.

Mechanically cleaned racks are generally erected almost vertically. Such bar screens have openings 25% in excess of the cross section of the sewage channel. Their area is usually half of that required for hand raked screens. Additional provision should be available for manual raking to take care of the situations where the mechanical rakes are temporarily out of order. Plants using mechanically cleaned screens have controls for (a) manual start and stop (b) automatic start and stop by clock control (c) high level switch (d) high level alarm (e) starting switch or overload switch actuated by loss of head and (f) overload alarm. The fabrication of screens should be such that bolts, cross bars, etc., will not interfere with raking operations.

5.6.1.2 Medium Screens

Medium bar screens have clear openings of 12 mm. Bars are usually 10 mm thick on the upstream side and taper slightly to the downstream side. These mechanically raked units are used before all pumps or treatment units such as the stabilization ponds. The bars used for the screens are rectangular in cross-section, usually about 10 mm × 50 mm and are placed with the larger dimension parallel to the flow. A weir on the side of the screen may be used as an overflow bypass.

5.6.1.3 Fine Screens

Fine screens are not normally suitable for raw sewage directly because of the clogging possibilities. They are mechanically cleaned devices using perforated plates, woven wire cloth or very closely spaced bars with clear openings of 5 mm or may be of the drum or disc type, mechanically cleaned and continuously operated.

Fine screens have generally a net submerged open area of not less than 0.05 m^2 for every 1000 m^3 of average daily flow of sewage from a separate system, the corresponding figure being 0.075 m^2 for combined systems. They are also used for beach protection where sewage without any further treatment is discharged into sea for disposal by dilution in situations where sewerage systems are not yet in place.

5.6.1.4 Comminuting Devices

A comminuting device is a mechanically cleaned screen which incorporates a cutting mechanism that shreds the retained material and enabling it to pass along with the sewage. The solids from the comminutor however, may lead to the production of more scum in the digester.

They are recommended for smaller sized STPs of up to 1 MLD.

5.6.1.5 Location of Screens

Screening devices are usually located where they are readily accessible because the nature of materials handled requires frequent inspection and maintenance of the installation. Where screens are placed in deep pits or channels, it is necessary to provide sufficiently wide approaches from the top and ample working space for easy access and maintenance.

Provision should be made for the-location of penstocks and bypass arrangements for the screens.

5.6.1.6   Housing of Screens

The need for a structure to house the screening equipment depends on two factors viz., the design of the equipment and the climatic conditions. If climatic conditions are not severe and could be withstood by the equipment, the screen housing can be omitted. Mechanically cleaned screens generally need suitable housing to protect the equipment, prevent accidents to operating personnel and improve the appearance of the STP. Ventilation of the housing is necessary to prevent accumulation of moisture and removal of corrosive atmosphere. An illustration is shown in Figure 5.21.

Figure 5.21   Mechanical screen with screen housing and ventilation at Koyambedu STP in Chennai. Manual screen and bypass channel screen in between the mechanical and manual screens is also seen

5.6.1.7   Hydraulics

A screen by its very nature and function collects material which will interfere with the flow. If the screen is cleaned continuously by mechanical arrangement, this interference will be kept to a minimum. Screens with intermittent cleaning arrangement are likely to produce surges of relatively high flow soon after cleaning. The usually accepted design is to place the base of the screen several centimetres below the invert of the approach channel and steepen the grade of the influent conduit immediately before the screen.

5.6.1.8   Velocity

The velocity of flow ahead of and through a screen varies materially and affects its operation. The lower the velocity through the screen, the greater is the amount of screenings that would be removed from sewage. However, the lower the velocity, the greater would be the amount of solids deposited in the channel. Hence, the design velocity should be such as to permit 100% removal of material of certain size without undue depositions. Velocities of 0.6 to 1.2 m/s through the open area for the peak flows have been used satisfactorily. When considerable amounts of storm water are to be handled, approach velocities of about 0.8 m/s are desirable to avoid grit deposition at the bottom of the screen, which might otherwise become inoperative when most needed during storm though lower value of 0.6 m/s is used in current practice.

Further, the velocity at low flows in the approach channel should not be less than 0.3 m/s to avoid deposition of solids. A straight channel before the screen is mandatory. Its length shall be a minimum of 5 times the width of the screen chamber. A similar channel after the channel is ideal for good hydraulics. Velocities can be got in the channel before the screen by adjusting the floor slope of the channel. These will ensure good velocity distribution across the screen and maximum effectiveness of the device.

5.6.1.9 Head Loss

Head loss varies with the quantity and nature of screenings allowed to accumulate between cleanings. The head loss created by a clean screen may be calculated by considering the flow and the effective areas of the screen openings, the latter being the sum of the vertical projections of the openings. The head loss through clean flat bar screens is calculated by the following formula:

$$h = 0.0729(V^2 - v^2)$$
(5.1)

where,

      h :   Head loss in m

      V :   Velocity through the screen in m/s

      v :   Velocity before the screen in m/s

Usually accepted practice is to provide loss of head of 0.15 m but the maximum loss with clogged hand cleaned screen should not exceed 0.3 m. For the mechanically cleaned screen, the head loss is specified by the manufacturers.

Another formula often used to determine the head loss through a bar rack is Kirschmer's equation:

$$h = \beta\,(W/b)^{4/3}\,h_v\,Sin\theta$$
(5.2)

where,

      h :   Head loss, in m

      β :   Bar shape factor which is assigned value of 2.42 for sharp edged rectangular bar, 1.83 for rectangular bar with semicircle upstream, 1.79 for circular, and 1.67 for rectangular bar with both u/s and d/s face as semi-circular

      W :   Maximum width of bar facing the flow, m

      b :   Minimum clear spacing between bars, m

      hv :   Velocity head of flow approaching rack, m

      θ :   Angle of inclination of rack with the horizontal

The head loss through fine screens is given by the formula

$$h = (1/2g)(Q/CA)^2 \qquad (5.3)$$

where,

> h :   Head loss, m
>
> Q :   Discharge, m^3/s
>
> C :   Coefficient of discharge (typical value 0.6)
>
> A :   Effective submerged open area, m^2

5.6.1.10 Quantity of Screenings

The quantity of screenings varies with the size of screen used and on the nature of sewage. Generally it has been found that for every the screenings from sanitary sewage vary from 0.0015 to 0.015 m^3 / million litres with screen sizes having clear opening of 100 mm and 25 mm respectively.

5.6.1.11 Other Screens

5.6.1.11.1 Rotary Drum Screens

Where the incoming sewage to the STP is at a higher elevation than ground level, the use of horizontal rotary screen is advantageous in that it avoids the need for manual scraping and the complicated forward and backward mechanical rakes. A typical drawing is shown in Figure 5.22.

1 Inlet

2 Head box

3 Wedge-wire drum

4 Doctor blade

5 High pressure spray

6 Drive motor
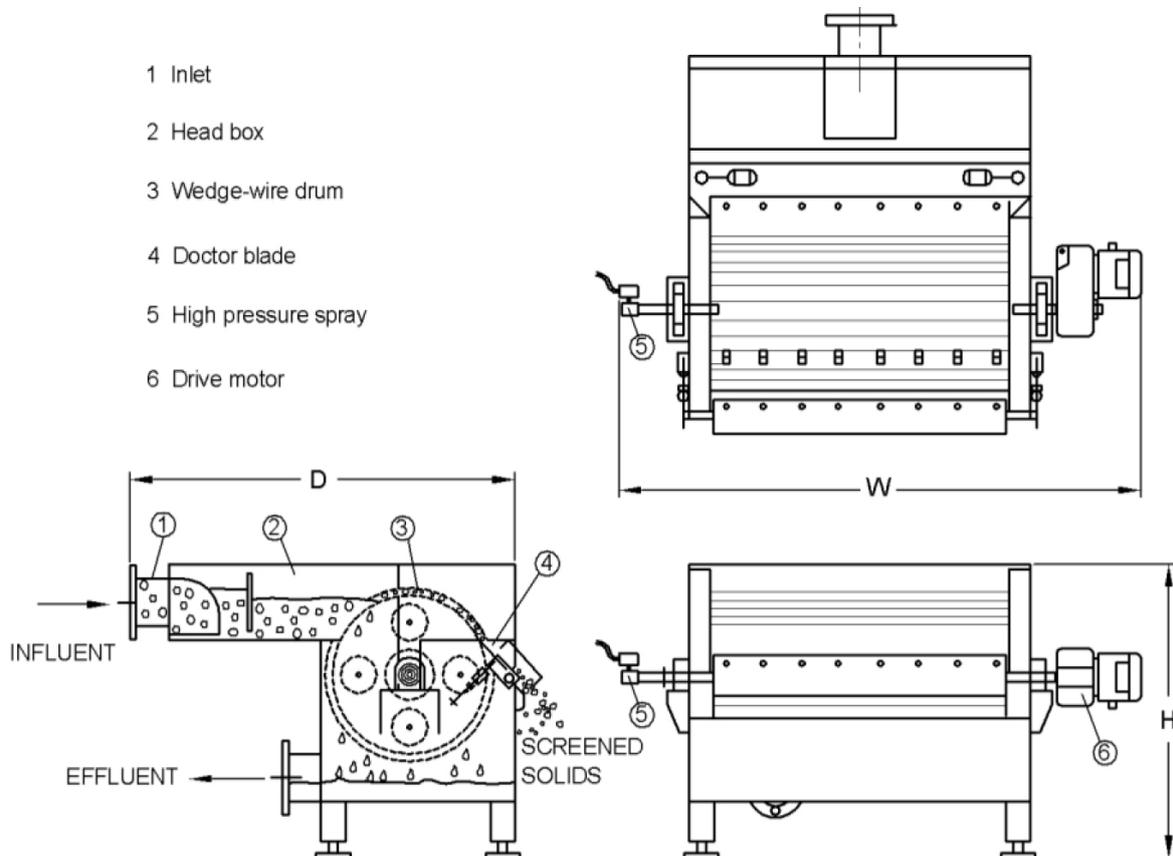
Figure 5.22 Typical rotary drum screen

The inlet sewage skims over the rotating screen and the screenings are intercepted and are rotated forward and to be are scraped onto conveyor belting. The screened sewage will go through the slots and fall downwards and get collected in a bottom trough and in the process it releases any sticking screenings to the circular screen in its downward rotation and the screenings float up for the screening on the upward movement. The screened sewage at the bottom can be collected in a channel and taken out in any suitable direction for downstream units. Though there are advantages of a simple operating system, there is the disadvantage of higher head loss compared to the bar screens due to the drop in sewage elevation for the height of the drum. Design criteria for these are vendor based and hence, these can be alternative offers in tender calls and then decided after evaluating the technical aspects. Their design criteria are generally as per the chosen manufacturer's design standards.

5.6.1.11.2   Circular Wedge Wire Screens

These are relatively simpler screens which operate on the principle shown in Figure 5.23
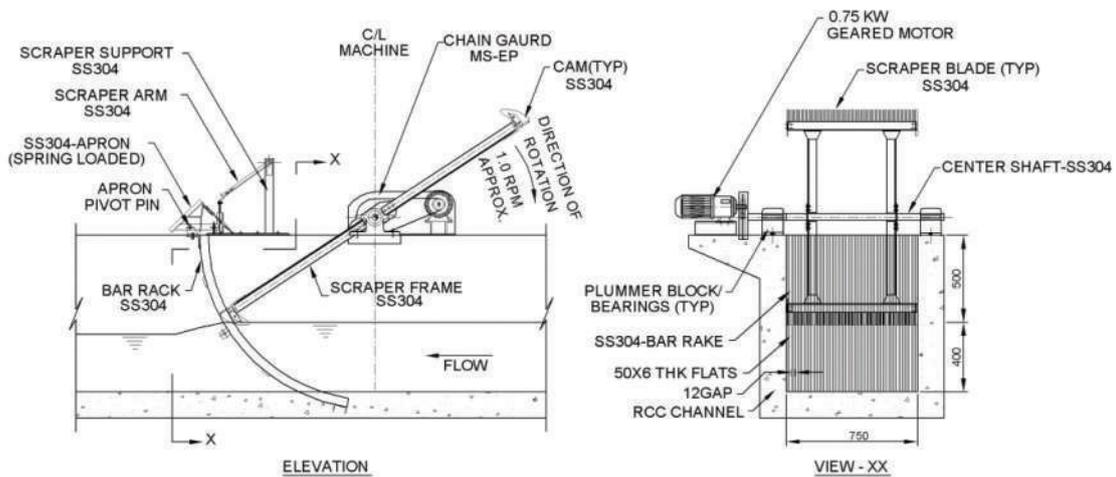
Figure 5.23 Typical circular wedge wire screen

The screen rods are usually wedge shaped which give a narrower slit in the upstream than the downstream slit of the screen openings and this prevents the clumping and jamming of the screenings while flowing across the slits. The screenings are continuously propelled by wire brushes on rotating arms and when they move up above the elevation of the screen rods, the screenings are ejected over a conveyor belt by the release of a spring loaded mechanism in the tubular arms holding the wire brush. The loss of head is just about the same as compared to the bar screens and the mechanism is simple for local servicing. Their design criteria are generally as per the chosen manufacturer's design standards. An installation is shown in Figure 5.24 overleaf.

5.6.1.12   Screen Compactor

The Screen Compactor is a unit which combines the following. The screen is usually a drilled or wedge wire metal screen section with mesh sizes that can be fabricated to suit and typically can be in the range of 0.25 mm to 10 mm.

Figure 5.24 UNIDO type circular wedge wire screen seen from upstream at left and from downstream at right in Ramanathapuram STP of Tamilnadu

Source TWAD Board, Chennai, Ramanathapuram STP

The conveyor is used to transmit the captured solids out of the sewage and dewater by gravity conveying the separated solids towards the pressing zone where it gets dewatered and compresses the screenings to reduce the volume. Their design criteria are generally as per the chosen manufacturer's design standards. This is shown in Figure 5.25.
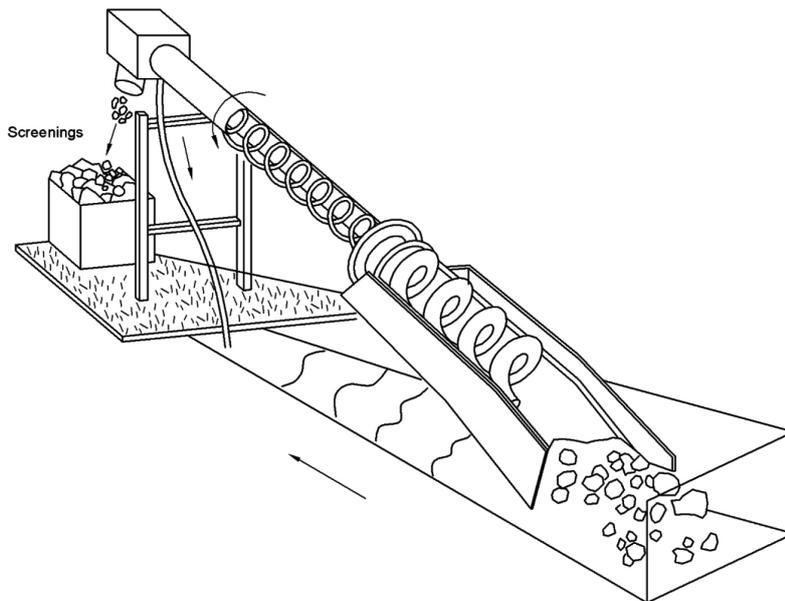
Figure 5.25 Typical screenings compactor

5.6.1.13 Screw Press

These are relatively more modern and are illustrated in Figure 5.26. The progressive compaction in the inclined conduit simultaneously, helps dewatering the screenings and compacting. Their design criteria are generally as per the chosen manufacturer's design standards.

Screw Press Cross Section
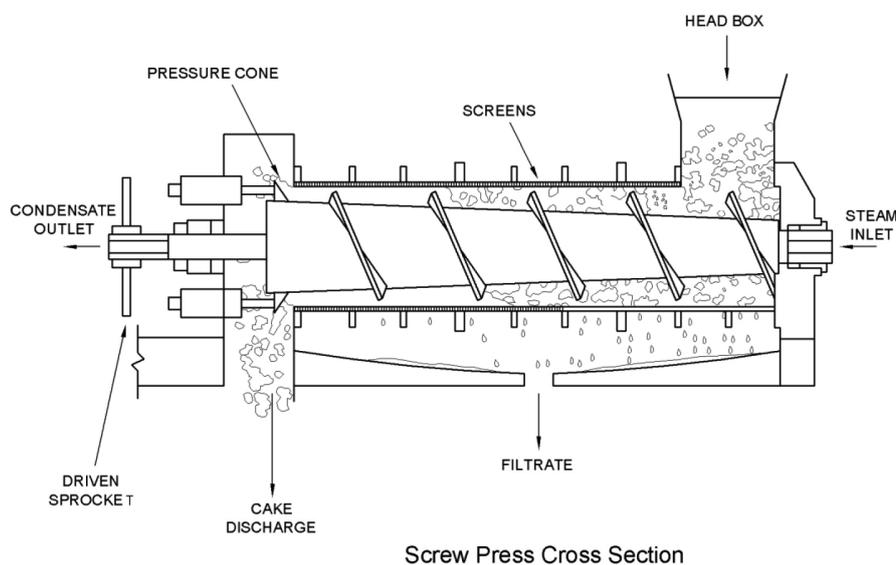
Figure 5.26 Typical screw press

5.6.1.14 Disposal of Screenings

The methods of disposal of screenings could be burial or composting. The screenings should not be left in the open or transported in uncovered conveyors as it would create nuisance due to flies and insects. If conveyors are used, they should be kept as short as possible for sanitary reasons. Burial in trenches usually 7.5 cm to 10 cm deep is practiced particularly in small installations.

At large works, where sufficient land for burial is not available within a reasonable distance from the plant, screenings shall be transported and mixed with town refuse for production of compost or for further processing and disposal as per guidelines / norms of the local PCB.

5.6.2 Grit Removal

Grit removal is necessary to protect the moving mechanical equipment and pump elements from abrasion and accompanying abnormal wear and tear. Removal of grit also reduces the frequency of cleaning of digesters and settling tanks. It is desirable to provide screens or comminuting device ahead of grit chambers to reduce the effect of rags and other large floating materials on the mechanical equipment, in case of mechanized grit chamber.

However, where sewers are laid at such depths as to make the location of grit chambers ahead of pumping units undesirable or uneconomical, only a bar screen is provided ahead of pumps, with grit chambers and other units following the pumps.

Accordingly, the sewage after removal of grit shall be as far as possible free of grit particles. The grit removal units should always have 100% standby.

5.6.2.1 Composition of Grit

Grit in sewage consists of coarse particles of sand, ash and clinkers, egg shells, bone chips and many inert materials inorganic in nature. Both quality and quantity of grit varies depending upon (a) types of street surfaces encountered (b) relative areas served (c) climatic conditions (d) types of inlets and catch basins (e) amount of storm water diverted from combined sewers at overflow points (f) sewer grades (g) construction and condition of sewer system (h) ground and ground water characteristics (i) industrial wastes (j) relative use of dumping chutes or pail depots where night soil and other solid wastes are admitted to sewers and (k) social habits. The specific gravity of the grit is usually 2.4 to 2.65. Grit is non-putrescible and possesses a higher hydraulic subsidence value than organic solids. Hence, it is possible to separate the gritty material from organic solids by differential sedimentation in a grit chamber.

5.6.2.2 Types

Grit chambers are of three major types as follows:

i) Velocity controlled V shaped long grit channels

ii) Square shaped chambers with entry and exit on opposite sides and mild hopper

iii) Vortex type cone and the centrifugal action plummets the grit to the bottom

They are mechanically cleaned and manually cleaned. The choice depends on several factors such as the quantity and quality of grit to be handled, head loss requirements, space requirements, topography and economic considerations with respect to both capital and operating costs. In very small plants, mechanization may be uneconomical. For STP flows of more than 10 MLD, mechanized grit removal units are preferred.

5.6.2.3 Vortex Type Units

The sewage is fed tangentially to induce a vortex type of flow, which will funnel the grit towards the centre, and hence be drawn down at the bottom chamber. An auxiliary agitator at this location keeps the grit in suspension and hence it is washed free of organics. The rim flow of the vortex is the degritted sewage to downstream units. The grit at the bottom can be either drained onto a grit filter bed by gravity or pumped to the beds depending on the levels. The filtrate is returned to the raw sewage. Though the centrifugal force and agitation are good controlling mechanisms, the additional head loss incurred in handling the filtrate and pumping if needed are comparatively avoided in the case of velocity controlled channels or detritors. This unit has its advantages in situations where sewage flow rates and durations vary widely. The hydraulic energy required for the vortex may compel the need to impart additional pumped energy to the sewage before degritting. A typical vortex type unit is shown in Figure 5.27 overleaf.

5.6.2.4 Vortex Type Units with Scum Removal

These are similar in their function to the vortex grit separator and have an additional provision to remove the scum. This is useful in situations where this problem is encountered in raw sewage.
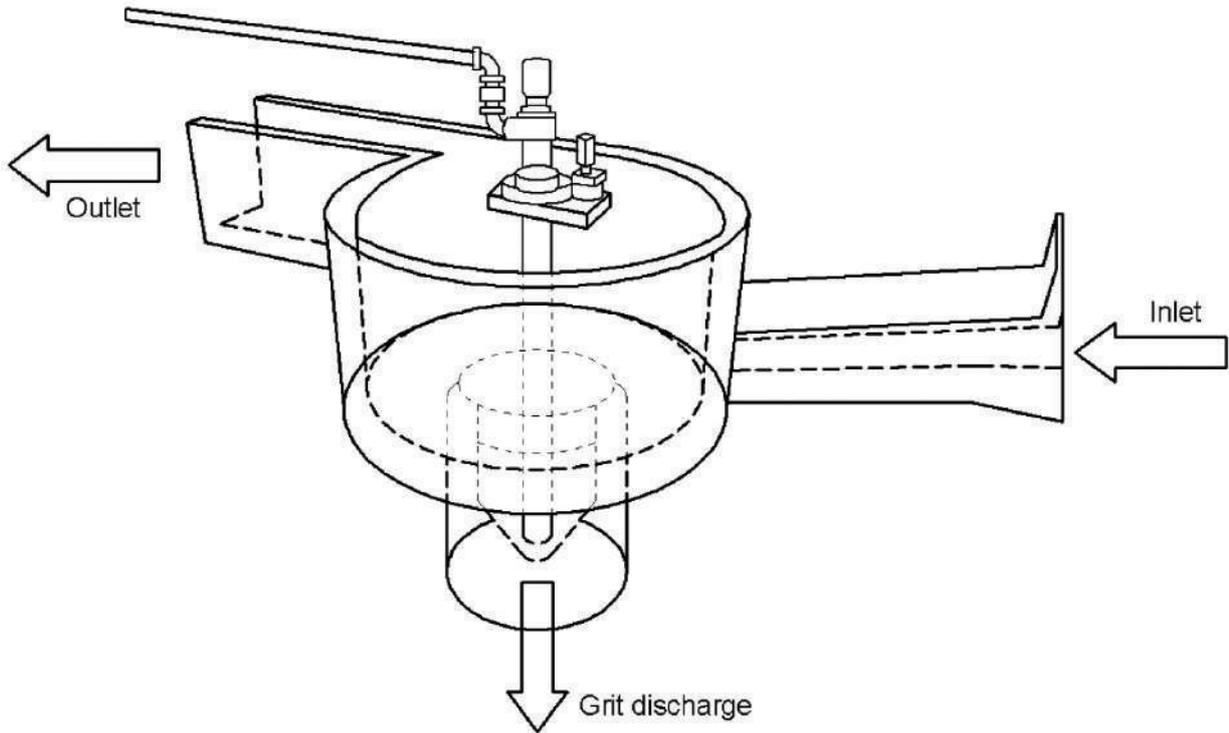
Figure 5.27 Hydraulic regimen of vortex grit separator

As otherwise, these are similar in their function and grit separation methods to vortex grit separator. A typical vortex type unit is shown in Figure 5.28.
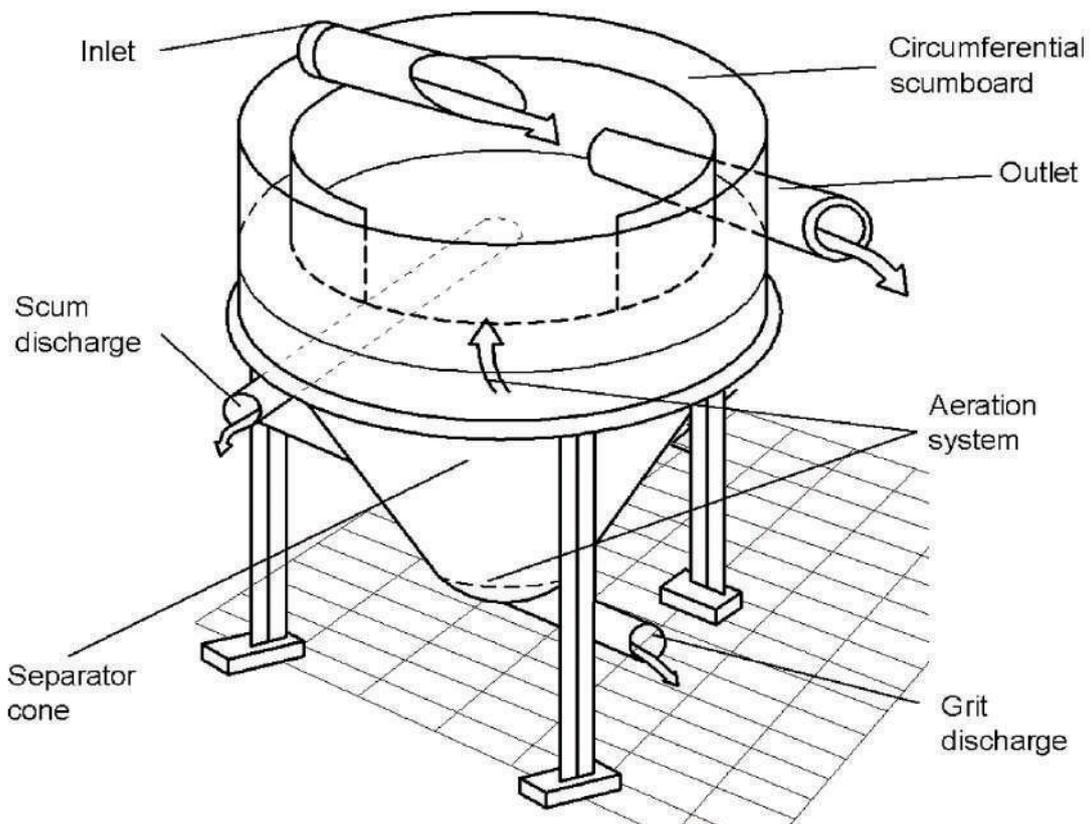
Figure 5.28 Hydraulic regimen of vortex and scum grit separator

5.6.2.5   Aerated Grit Chambers

An aerated grit chamber is a special form of grit chamber consisting of a standard spiral-flow aeration tank provided with air-diffusion tubes placed on one side of the tank, 0.6 to 1 m from the bottom. The grit particles tend to settle down to the bottom of the tank at rates dependent upon the particle size and the bottom velocity of roll of the spiral flow. This is in turn controlled by the rate of air diffusion through the diffuser tubes and the shape of the tank. The heavier grit particles with their higher settling velocities drop down to the floor whereas the lighter organic particles are carried with roll of the spiral motion and eventually out of the tank. The velocity of roll, however, should not exceed the critical velocity of scour of grit particles. Normally, a transverse velocity of flow, not exceeding 0.4 m/s to 0.6 m/s at the top of the tank should satisfy this requirement for differential scour. No separate grit washing mechanism or control device for horizontal velocity is necessary in aerated grit chambers. A typical drawing is shown in Figure 5.29. The aerated de gritting removals are reproduced from WEF MOP 11 hereunder.

Transverse velocity at surface	0.6-0.8 m/s
Depth-to-width ratio	1.5:1 to 2:1
Air supply	4.6-7.7 l/m/s of length
	0.3-0.4 m^3 /m^3
Detention time	3-5 min (peak)
Quantity of grit	7.5-75 ml/m^3
Quantity of scum (skimmings)	7.5-45 ml/m^3
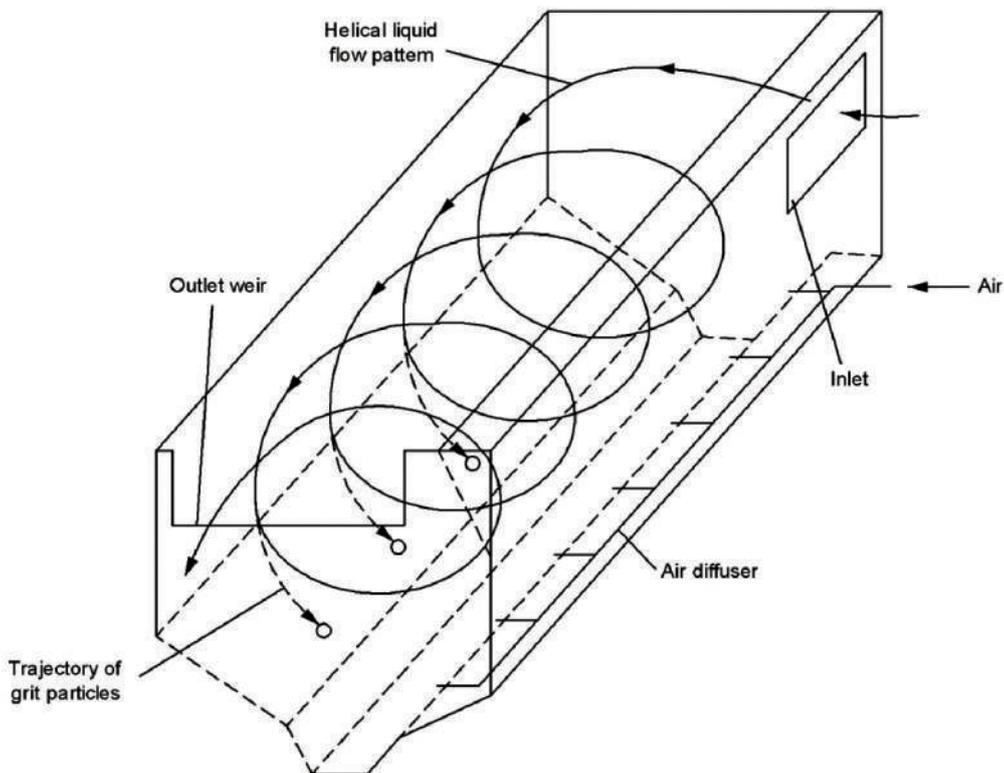
Source: EPA

Figure 5.29 Aerated grit chamber

5.6.2.6   Clearing of the Grit

Manual clearing of the grit is to be avoided except in the case of very small STPs of less than 1 MLD where velocity controlled channels can be cleared by the operator using a shovel and walking on a platform along the length. Mechanical clearing is advocated and these are provided with mechanical equipment for not only collection but also washing of grit and can be operated on either a continuous or an intermittent basis. Scraper blades or ploughs rotated by a motor drive, collect the grit settled on the floor of the grit chamber. The grit so collected is elevated to the ground level by several mechanisms such as bucket elevators, jet pump, screws and air lift. The grit washing mechanisms are also of several designs most of which are agitation devices using either water or air to produce a washing action. In intermittently (normally once or twice a day) operated type, sufficient storage capacity to hold the grit between intervals of grit elevation should be provided.

5.6.2.7   Design Data

The basic data essential for a rational approach to the design of grit chambers are hourly variations of sewage flow and typical values for minimum, average and peak flows. Since the grit chamber is designed for peak flows and the flow through velocity is maintained constant within the range of flow, successful design and operation of grit chamber calls for a fairly, accurate estimation of the flows. The quantity and quality of grit varies from sewage to sewage. Data relating to these two factors is very useful in proper design of grit collecting, elevating and washing mechanisms. In the absence of specific data, grit content may be taken as 0.05 to 0.15 m³/million litres for sewage and 0.06 to 0.12 for combined sewage. The quantity may increase three to four fold during peak flow hours, which may last for 1 to 2 hours.

5.6.2.7.1   Generic Design of Grit Chambers

5.6.2.7.1.1   Settling Velocity

Grit chamber may be designed on a rational basis by considering it as a sedimentation basin. The grit particles are treated as discrete particles settling with their own settling velocities. The settling velocity is governed by the size and specific gravity of the grit particles to be separated and the viscosity of the sewage. The minimum size of the grit to be removed is 0.20 mm although 0.10 to 0.15 mm is preferred for conditions where considerable amount of ash is likely to be carried in the sewage. The specific gravity of the grit may be as low as 2.4, but for design purposes a value of 2.65 is used. The settling velocity of discrete particles can be determined using the appropriate equation depending upon the Reynolds number,

1. Stoke's Law

$$V_s = \frac{g}{18} \frac{(\rho_s - \rho)}{\rho} \frac{d^2}{v} \qquad (5.4)$$

or

$$= \frac{g}{18}(S_s - 1)\frac{d^2}{v}$$

where,

V_s :   Settling velocity, m/s

g :   Acceleration due to gravity, m/s^2

ρ :   Mass density of grit particle, kg/m^3

ρ :   Mass density of liquid, kg/m^3

d :   Size of the particle, m

v :   Kinematic viscosity of sewage, m^2/s

S_s :   Specific gravity of grit particle, dimensionless

Stoke's law holds good for Reynolds number R below 1.0;

$$R = V_s d / v$$

For grit particles of specific gravity of 2.65 and liquid temperature at 10 degree;

$$v = 1.01 \times 10^{-6} m^2 / s$$

This corresponds to particles of size less than 0.1 mm. The flow conditions are laminar where viscous forces dominate over inertial forces.

2. Transition Law

The design of grit chamber is based on removal of grit particles with minimum size of 0.2 mm or 0.15 mm and therefore Stoke's Law is not applicable to determine the settling velocity of the grit particles for design purposes.

The settling velocity of a discrete particle is given by the general equation

$$V_s = \sqrt{\frac{4}{3} \frac{g}{C_D} \frac{(\rho_s - \rho)}{\rho}} \, d \tag{5.5}$$

Where C_D is the Newton coefficient of Drag which is a function of Reynolds number. The transition flow conditions hold when Reynolds number is between 1 and 1,000. In this range, C_D can be approximated by

$$C_D = \frac{18.5}{R^{0.6}} = \frac{18.5}{\left(\dfrac{V_s d}{v}\right)^{0.6}} \tag{5.6}$$

Substituting the value of C_D in equation (5.6) and simplifying

$$V_s = \left[0.707(S_s - 1)d^{1.6}v^{-0.6} \right]^{0.714} \tag{5.7}$$

The settling velocity of grit particles in the transition zone is also calculated by the Hazen's modified formula

$$V_s = 60.6(S_s - 1)d\frac{3T + 70}{100} \tag{5.8}$$

Where d in equation (5.8) is in cm and T is the temperature in degree Centigrade and Vs in cm/s.

The settling velocity of grit particles in the range of 0.1 mm and 1mm can be determined using equation (5.7) and this equation or its approximate empirical form of equation (5.8) should be used in design of grit chambers which are designed to remove particles of size 0.15 mm or 0.2 mm.

3. Newton's Law

When the particle size increases beyond 1 mm and Reynolds number beyond 1,000, the Newton coefficient drag C_D assumes a constant value of 0.4 and the following equation can be used to determine the settling velocity of grit particles.

$$V_s = \left[3.3g(S_s - 1)d \right]^{0.5} \tag{5.9}$$

5.6.2.7.1.2 Surface Overflow Rate

Efficiency of an ideal settling basin is expressed as the ratio of the settling velocity of the particles to be removed (V_s) to the surface overflow rate (V_o).

$$\eta = \frac{V_s}{V_o} \tag{5.10}$$

Where V_o is defined as the ratio of flow of sewage to be treated in an ideal settling tank to the plan area of the tank, i.e., Q/A. It is equal to the settling velocity of those particles which will be 100% removed in an ideal settling tank.

In an ideal settling basin, all particles having settling velocity, Vs ≥ Vo are completely removed. However, particles having settling velocity, Vs < Vo are removed in proportion to the ratio of V_s to V_o.

Table 5.6 gives settling velocity of different size particles of specific gravity 2.65 (inorganic grit particles) and corresponding surface overflow rates for 100% removal of these particles based on Equation (5.8).

Though the different settling patterns occur as described here, for purpose of preparing DPRs, the discreet settling alone shall be considered to simplify the computations and the criteria is shown in Table 5.6.

Table 5.6 Settling velocities and surface overflow rates for ideal grit chamber at 10ºC

Diameter of Particles, mm	Settling velocity m/s,	Surface Overflow rate $m^3 / d /m^2$
	Ss = 2.65	Ss = 2.65
0.20	0.025	2160
0.15	0.018	1555

Source: CPHEEO, 1993

However, the behaviour of a real grit chamber departs significantly from that of the ideal settling basin due to turbulence and short-circuiting resulting from eddy, wind and density currents. Hence, the surface overflow rates (SOR) should be diminished to account for the basin performance. Following equation could be used to determine the SOR for a real basin for a given efficiency of grit removal and basin performance.

$$\eta = 1 - \left[1 + nV_s/(Q/A)\right]^{-\frac{1}{n}} \qquad (5.11)$$

where,

η :   Desired efficiency of removal of grit particle

V_s :   Settling velocity of minimum size of grit particle to be removed

Q/A :   Design surface over flow rate applicable for grit chamber to be designed

n :   An index which is a measure of the basin performance.

The values of n are 1/8, 1/4, 1/2 and 1 for very good, good, poor and very poor performance. It can be seen that the design surface overflow rate will be 66.67%, 58.8%, 50% and 33.3% of the settling velocity of the grit particles to be removed to achieve 75% removal efficiency in grit chamber with very good, good, poor and very poor tank performance respectively. In practice, values of two thirds to one half are used in design depending upon the type of the grit chamber.

5.6.2.7.1.3   Detention Period

Once the area is calculated surface overflow rate and the liquid depth is ascertained from the equipment manufacturer, the resulting volume and hence detention time at average flow shall be checked up as not to exceed 60 seconds.

5.6.2.7.1.4   Bottom Scour and Flow through Velocity

Bottom scour is an important factor affecting grit chamber efficiency and the scouring process itself determines the optimum velocity of flow through the unit. This may be explained by the fact that there is a critical velocity of flow (V_c) beyond which particles of a certain size and density once settled, may be again placed in motion and reintroduced into the stream of flow. The critical velocity for scour may be calculated from modified Shields' formula:

$$V_c = K_c \sqrt{g(S_s - 1)d} \qquad (5.12)$$

where K_c = 3 to 4.5. A value of 4.0 is usually adopted for grit particles.

For a grit particle size of 0.2 mm, the formula gives critical velocity values of 17.1 to 25.6 cm/sec. In actual practice; a horizontal velocity of flow of 15 to 30 cm/sec is used at peak flows. The horizontal velocity of flow should be maintained constant at other flow rates also to ensure that only organic solids and not the grit are scoured from the bottom. Bottom scour is an important factor particularly affecting the grit chamber efficiency. Design example of velocity controlled grit chamber and detritors are shown in Appendix A.5.7 and A 5.8

5.6.2.7.2   Specific Design of Recent Devices

Their design criteria are generally as per the chosen manufacturer's design standards.

5.6.2.7.3   Velocity Control Devices for Grit Channels

In earlier days, velocity controlled grit chambers were used which needed constant velocities at all depths of flow and for this purpose, devices as proportional weir and Sutro weirs were used downstream. Numerous devices have been designed in an attempt to maintain a constant horizontal velocity of flow through grit chambers in the recommended range of 15 to 30 cm/sec. A variation of 5 to 10% above and below the desired velocity is permitted. Multiple channels with the total capacity to carry the maximum flow are to be adopted.

A satisfactory method of controlling the velocity of flow through the grit channels is by using a control section at the end of the channel, This varies the cross sectional area of flow in the section in direct proportion to the flow.

As for example, for a flow of 5 cum/s the cross-sectional area of flow should be 5 m2 and when flow decreases to 3 cum/s the cross-sectional area of flow should be reduced to 3 m2 to maintain the same velocity in the channel. Such control sections include proportional flow weirs, Sutro weirs and Parshall flumes. In practice, the Parshall flumes are commonly used.

a) Proportional Flow Weir

The proportional flow weir is a combination of a weir and an orifice as shown in Figure 5.30. It maintains a nearly constant velocity in the grit channels by varying the cross-sectional area of flow through the weir so that the depth is proportional to flow.
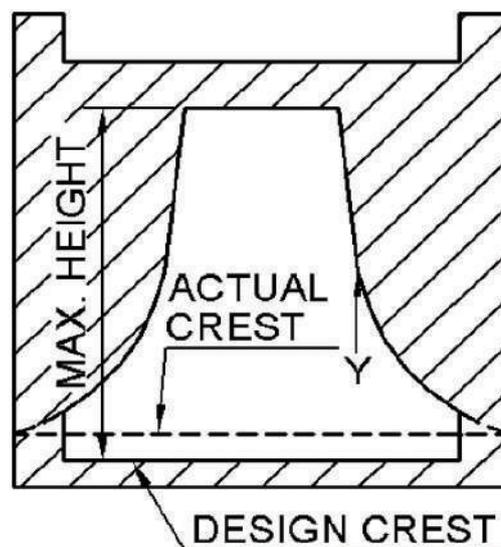
Source: CPHEEO, 1993

Figure 5.30 Proportional flow weir

The general equation for determining the flow through weir, Q is

$$Q = cb\sqrt{2ag}\left(H - \frac{a}{3}\right)$$ (5.13)

where,

c : Coefficient which is assumed 0.61 for symmetrical sharp-edge weirs.

a : Dimension of weir usually assumed between 25 mm and 50 mm

b : Base width of the weir

H : Depth of flow

To determine the shape of the curve forming the outer edges of the cut portion, the following equation of curve forming the edge of the weir may be used.

$$x = \frac{b}{2}\left(1 - \frac{2}{\pi}\tan^{-1}\sqrt{\frac{Y}{a} - 1}\right)$$ (5.14)

where,

x : Weir width at liquid surface

Y : Liquid depth

The weir shall be set from 100 mm to 300 mm above the bottom of grit chamber to provide grit storage or for operation of mechanical grit clearing.

The weir should also be set at such an elevation as to provide a free fall into the outlet channel as it cannot function under submerged conditions. Each grit chamber should be provided with a separate control weir. An installation is shown in Figure 5.31.

Figure 5.31 Typical Installation of proportional flow weir.

Photo at left is view from the grit channel side. Photo at right is the outlet from the weir showing the graduated readings at various depths of flows

Source-Dr Absar Kazmi, Training Course by Department of Civil Engineering, IIT Roorkee

b) Sutro Weir

The Sutro-weir is a half proportional flow weir cut symmetrically and centrally along the vertical axis as illustrated in Figure 5.32. The orifice has a straight horizontal bottom forming the weir.
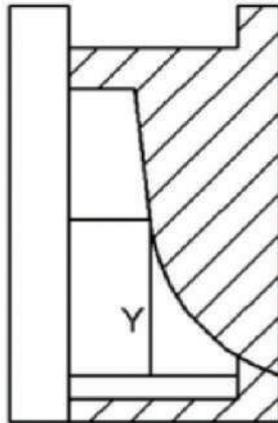
Source: CPHEEO, 1993

Figure 5.32 Sutro weir

Grit removal from these channels is normally carried out by suction pumps mounted on a bridge travelling the full length in both forward and return directions.

The suction end of these pumps has a hose suspended into the sewage in the channel. They discharge through a header into a channel built along the side of the grit channel. Most often, the suction fails and the grit is not removed. Hence, these mechanized grit removal systems and the velocity controlled long grit channels are proposed to be phased out.

5.6.2.8   Flow Measurement

Flow measurement is invariably to be provided for at the downstream of the grit removal facilities. The Parshall flume is normally used.

5.6.2.8.1   Parshall Flume

A Parshall flume is an open constricted channel, which can be used both as a measuring device and also as a velocity control device. It is more commonly used for the latter purpose in downstream of grit chambers.

The flume has a distinct advantage over the proportional flow weir, as it involves negligible headloss and can work under submerged conditions up to certain limits as in Figure 5.33 overleaf.

The limits of submergence are 50% in case of 150 mm throat width and 70% for wide throat widths up to 1 m as in Table 5.7 overleaf.

Another advantage is that one control section can be installed after the exit channel of a number of parallel grit chambers. to measure the total flows through all the grit channels.

PLAN

LONGITUDINAL SECTION
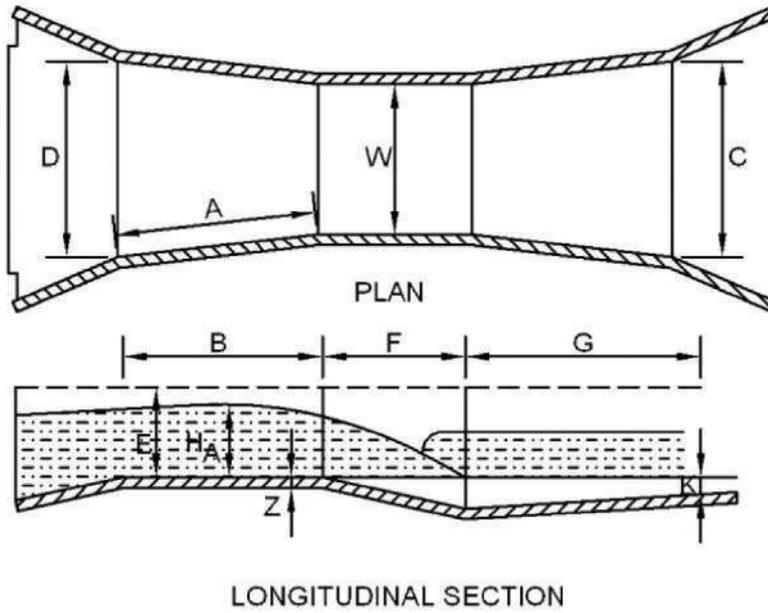
Source: CPHEEO, 1993

Figure 5.33 Dimensions for Parshall flume

Table 5.7 Dimensions of Parshall flume (mm)

Flow range Q max (mld)[+]	W	A	B[++]	C	D[+++]	F	G	K	Z
Up to 5	75	460	450	175	255	150	300	25	56
5-30	150	610	600	315	391	300	600	75	113
30 - 45	225	865	850	375	566	300	750	75	113
45-170	300	1350	1322	600	831	600	900	75	225
170-250	450	1425	1397	750	1010	600	900	75	225
250 -350	600	1500	1472	900	1188	600	900	75	225
350- 500	900	1650	1619	1200	1547	600	900	75	225
500- 700	1200	1800	1766	1500	1906	600	900	75	225
700-850	1500	2100	2060	2100	2625	600	900	75	225
850-1400	2400	2400	2353	2700	3344	600	900	75	225

+   For average flow and peak factors, see earlier sections

++   Value should be equal to $1.5 \times (Qmax)^{1/3}$ but not less than those shown in the Table

+++   For higher values of B (than shown in the table), the values of D also to be increased to keep D/B ratio same as in table

Source: CPHEEO, 1993

The flume is also self cleansing and there is no problem of clogging. As the Parshall flume is a rectangular control section, the grit chamber above it must be designed to approach a parabolic cross section. However, a rectangular section with a trapezoidal bottom may be used with a Parshall flume in which case the variations in velocity at maximum and minimum flow conditions from the designed velocity of flow should be within permissible limits as given by the following equations.

$$Q = 2264\, W \left(H_A\right)^{\frac{3}{2}} \tag{5.15}$$

$$D + Z = 1.1 H_A \tag{5.16}$$

and

$$\frac{D_{min}}{D_{max}} = \frac{1.1\left(\dfrac{Q_{min}}{2264W}\right)^{2/3} - Z}{1.1\left(\dfrac{Q_{max}}{2264W}\right)^{2/3} - Z} \tag{5.17}$$

$$D = 1.1\left(\frac{Q}{2264W}\right)^{2/3} - Z \tag{5.18}$$

$$b = \frac{Q_{max}}{1000\, D_{max} V_{max}} \tag{5.19}$$

or

$$b = \frac{Q_{min}}{1000\, D_{min} V_{min}} \tag{5.20}$$

$$V = \frac{Q}{1000\, bD} \tag{5.21}$$

where,

Q :	Rate of flow in lps
Q_{min} :	Minimum rate of flow in lps
Q_{max} :	Maximum rate of flow in lps
W :	Throat width in m
H_A :	Depth of flow in upstream leg of the flume at one third point in m
Z :	Constant in m
D :	Depth of flow in grit chamber in m
b :	Width of grit chamber in m
V :	Velocity of flow in m/s at particular depth of flow

Recommended throat widths for different ranges of flow along with the dimensions of the various dimensions of the flume for the different throat widths are given in Table 5.7 which should be strictly adhered to. A typical example is shown in Appendix A.5.9. A typical Parshall flume installation is shown in Figure 5.34

A Parshall flume in use showing the ultrasonic level measurement equipment mounted over the flume at the throat.

Source-Dr Absar Kazmi, Training Course by Department of Civil Engineering, IIT Roorkee

Figure 5.34 Typical Parshall Flume Installation

5.6.2.8.2   Number of Units

In case of manually cleaned grit chambers at least two units should be provided. All mechanically cleaned units should be provided with a manually cleaned unit as standby.

5.6.2.8.3   Dimensions of Each Unit

The surface areas required for each unit is worked out based on the overflow rate. In case of mechanized grit chambers, the plan dimension and liquid depth shall be readjusted to suit the standard sizes of the mechanical equipment. Additional depth for storage of grit between intervals of cleaning shall be provided depending on the interval of cleaning. A free board of 150 mm to 300 mm shall be provided. Bottom slopes are based on the type of scraper mechanism used.

5.6.2.8.4   Loss of Head

Loss of head in a grit chamber varies from 0.06 m to 0.6 m depending on the device adopted for velocity control in velocity controlled grit chambers. In mechanized units, the free fall over the exit weir shall not be less than 200 mm at peak flow.

5.6.2.9   Disposal of Grit

Clean grit is characterized by the lack of odours. Washed grit may resemble particles of sand and gravel, interspersed with particles of egg-shell, and other similar relatively inert materials from the households. Grit washing mechanism has to be included whenever the detention time is more and flow through velocity is less. Unless washed, it may contain considerable amount of organic matter. This becomes an attraction to rodents and insects and is unsightly and odorous. The grit may be disposed of by dumping or burying or by sanitary landfill. The ultimate method used however depends upon the quantity and characteristics of the grit, availability of land for dumping, filling, or burial. In general, unless grit is washed, provision for burial should be made.

5.6.2.10   Choice of Unit

The square detritor offers perhaps the reasonable preferred combination of (a) mechanical degritting, (b) mechanical grit washing, (c) minimum head loss, (d) grit delivery at a required elevation to stationed trucks beneath it, (e) slow moving central drive and grit classifiers with minimum wear and tear and (f) all these without the need for manual labour to handle the grit. The grit classifier drive system is to be preferred in mild steel appropriately coated as compared to cast iron cam drives because these cast iron parts will need long intervals to be sent back to the foundry for repairs. Here again, the classifiers can be either the screw or the to & fro raker.

The screw type will need a factory made stainless steel trough to match the screw profile. It has the advantage of a flappable semi cylindrical hinged cover and preventing odours and insects around it and protection in rains. The raker type can be accommodated in ordinary masonry channel. Their choice is a matter of preference by the user agency.

5.6.3   Flow Equalization

The design criteria given in the manual for the treatment portion are based on the DWF. There is no need to make changes to these for peak flows. The necessary adjustments are in-built in these design criteria.

However, when the peak factor exceeds 3 by a wide margin, it is advisable to equalize the sewage flow before feeding to the STP units. This can be either by inline equalization tank or side stream flow balancing tank depending on the volume of the raw sewage flow. The illustrations in Section 3.18 using side stream leaping weirs or floor level leaping weirs can be used to allow the average flow to go on to the STP and excess flows to spill into a balancing tank from where it can be supplemented by pumping during lean flow periods. However, the option of providing either the inline equalization tank or side stream flow balancing tank is decided based on the specific situation and cost economics. The volume of such equalization tanks are to be based on the mass diagram. An example for Chennai sewage conditions as measured in summer and monsoon seasons is illustrated as Appendix A.5.2.

5.7   SETTLING

5.7.1   General

The words settling tanks, sedimentation tanks and clarifiers are used in various manuals and text books. All these mean the same. A more popularly used name is clarifiers. These are used to separate the suspended solids, which can settle by gravity when the sewage is held in a tank. If these suspended solids are discharged into water courses, they will result in sludge banks. If these are used for land disposal, it will lead to clogging of soil pores and uncontrolled organic loading.

The primary clarifier is located after screens and grit chambers and reduces the organic load on secondary treatment units. It is used to remove (i) inorganic suspended solids or grit if it is not removed in grit chamber described earlier, (ii) Organic and residual inorganic solids, free oil and grease and other floating material and (iii) chemical flocs produced during chemical coagulation and flocculation.

Secondary clarifier is located after the biological reactor and is used to separate the bio-flocculated solids or bioflocs of biological reactors. In some cases where two stage bio reactors are used, the clarifiers after the first stage of bioreactor is referred to as intermediate clarifiers.

Septic tanks, Imhoff tanks and clarigester are combination units where digestion of organic matter and settling are combined in the same unit and is meant for small installations.

Settling also occurs in waste stabilization ponds and facultative aerated lagoons. However, the settled organic matter is stabilized in the pond itself and no separate unit is provided.

5.7.2   Characteristics of Settleable Solids

The settleable solids to be removed in primary or secondary clarifiers are mainly organic and flocculent in nature. These are either dispersed or flocculated. Their specific gravity varies from 1.01 to 1.02. The bulk of the finely divided organic solids reaching primary clarifiers are low specific gravity solids which are incompletely flocculated but are susceptible to flocculation. Flocculation occurs within primary clarifiers due to eddying motion of the fluid. The aggregation becomes more complete as the sewage is detained for longer periods (hydraulic residence time) in these tanks. However, the rate of flocculation rapidly decreases as the detention period is increased beyond certain values. Hence prolonged detention periods are not productive and in fact may be counter productive by inducing septic conditions and generation of sulphide gas.

5.7.3   Types of Settling

Mainly , four categories of settling occur, depending on the tendency of particles to interact and their concentration. These settling types are (i) Discrete settling (ii) Flocculent settling (iii) Hindered or zone settling (iv) Compression.

5.7.3.1   Discrete Settling

Discrete particles do not change their size, shape or mass during settling. Grit in sewage behaves like discrete particles. The settling velocity of discrete particles is determinable using Stokes or Transition law. Organic solids in raw sewage and bioflocs in biologically treated sewage cannot be considered as discrete particles and hence Stoke's law is not applicable for these particles.

5.7.3.2   Flocculent Settling

Flocculent particles coalesce during settling, increasing the mass of particles and settle faster. Flocculent settling refers to settling of flocculent particles of low concentration usually less than 1000 mg/l. The degree of flocculation depends on the contact opportunities, which in turn are affected by the surface overflow rate, the depth of the basin, the concentration of the particles, the range of particle sizes and the velocity gradients in the system. No adequate mathematical equation exists to describe flocculent settling and therefore, overflow rates to achieve a given removal efficiency are determined using data obtained from settling column studies. The removal of raw sewage organic suspended solids in primary settling tanks, settling of chemical flocs in settling tanks and of bioflocs in the upper portion of secondary sedimentation tanks are examples of flocculent settling.

5.7.3.3   Hindered or Zone Settling

When concentration of flocculent particles is in intermediate range, they are close enough together so that their velocity fields overlap causing hindered settling. The settling of particles results in significant upward displacement of water. The particles maintain their relative positions with respect to each other and the whole mass of particles settles as a unit or zone. This type of settling is applicable to concentrated suspensions found in secondary settling basins following activated sludge units. In the hindered settling zone, the concentration of particles increases from top to bottom leading to thickening of the settled particles at the bottom. Such secondary clarifiers where zone settling occurs are designed based on solids loading for the given area of the water surface. The required loading rate can be determined by conducting settling column analysis in the laboratory. However, the values of best design are readily given in this manual in section 5.7.

5.7.3.4   Compression

In compression zone, the concentration of particles becomes so high that particles are in physical contact with each other, the lower layers supporting the weight of upper layers. Consequently, any further settling' results due to compression of the whole structure of particles and accompanied by squeezing out of water from the pores between the solid particles. This settling phenomenon occurs at the bottom of deep sludge mass, such as in the bottom of secondary clarifiers following secondary biological treatment and in tanks used for thickening of sludge.

5.7.4   Design Considerations

5.7.4.1   Factors Influencing Design

Several factors such as flow variations, density currents, solids concentration, solids loading, area, detention time and overflow rate influence the design and performance of clarifiers. In the design of some plants, only a few of these factors may have significant effect on performance while in others, all of them may play an important role. Clarifiers are designed for average flow conditions. Hence, during peak flow periods, the detention period gets reduced with increase in the overflow rate and consequent overloading for a short period. If hourly flow variations are wide, a flow equalization tank will be useful before the treatment units so that uniform hydraulic loading is possible.

5.7.4.2   Design Criteria

The design criteria shall consist of surface loading rate, solids loading rate, weir overflow rate and side water depth.

5.7.4.2.1   Surface Loading Rate

This represents the hydraulic loading per unit surface area of tank in unit time expressed as $m^3/d/m^2$ and must be checked, both, for average flow and peak flow.

5.7.4.2.2   Solids Loading Rate

The solids loading rate is an important decision variable for the design of secondary clarifier which settles the bio-flocculated solids. It is expressed as kg SS/d/m^2.

5.7.4.2.3   Weir Loading

Weir loading influences the removal of solids particularly in secondary clarifiers. There is no positive evidence that weir loading has any significant effect on removal of solids in primary clarifiers. However, certain loading rates based on practice are recommended both for primary as well as secondary clarifiers. The loading should however ensure uniform withdrawal over the entire periphery of the tank to avoid short-circuiting or dead pockets. Performance of existing clarifiers for SS removal can be improved by merely increasing their weir length.

Primary and secondary clarifiers normally have the V notch at the weir overflow rim. The CMWSSB is operating a 23 MLD STP using conventional ASP with 2 primary clarifiers of each 21.2 m diameter and 2 secondary clarifiers of each 24.4 m diameter. Their RCC sidewall is topped with 14 cm thick brick pillars of 23 cm length interspaced with masonry bevelled weirs of 70 cm length as in Figure 5.35.
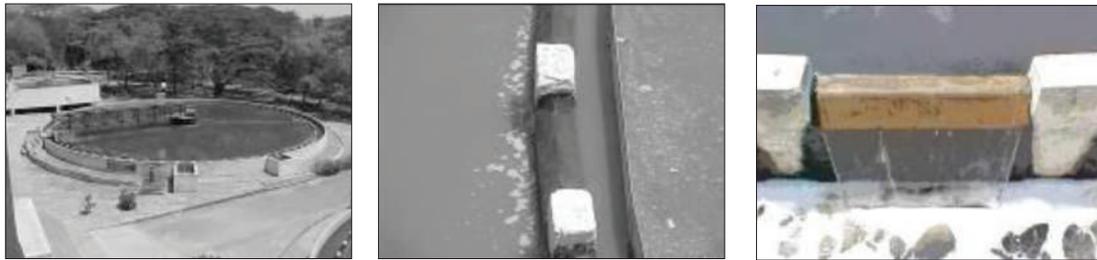
Figure 5.35 Masonry Bevelled Weirs of Primary Clarifiers (left & centre) and Secondary Clarifier (right) with additional entrainment aeration of the treated sewage by the freefall over the weir.

In construction, it is easy to guide the mason by a levelling instrument to finish these weirs all at the same elevation as he has to trowel the small length between the pillars one at a time. This arrangement has not resulted in any corrosion and facilitates easy cleaning of the lip and overflow face of the weir daily. In terms of weir length, this effectively means the weir length is 70/93 = 75% of the peripheral length. The weir loading rate at average flow works out to 23,000/(2×3.14×21.2×0.75) = 230 cum/m/day for primary clarifier and 23,000/(2×3.14×24.4×0.75) = 200 cum/m/day for secondary clarifier., though the manual guidelines are limited to 125 and 185 respectively. The suspended solids in overflow in these secondary clarifiers are consistently between 20 and 30 mg/l. Though these higher weir overflow rates are reportedly functioning well, still, complying with the reduced loading rates of 125 and 185 can only prevent the drag of SS over the weirs and better SS removals. However, if v notched weirs are preferred they can be used with appropriate material of weir plates and fasteners.

5.7.4.2.4   Depth and Detention Time

Once the surface area is arrived at from overflow rate and solids loading rate, the next step is the determination of the depth which influences the detention time and vice versa. The depth considered for design is the vertical side water depth (SWD). It influences the hydrostatic compression of the bottom sludge solids. Thus deeper depths will give higher concentration in the sludge solids withdrawn from the bottom sludge pit. Shallow depths will result in loose solids concentration.

This will require huge volumes of wet sludge to be withdrawn for taking out a given weight of sludge solids. In turn, these volumes have a heavy bearing in the required volumes of the sludge handling units and their associated piping and valves etc. Hence drawal of dense sludge is more beneficial.

In the case of secondary clarifiers, another issue is that longer residence times may result in all the residual dissolved oxygen in the treated sewage being fully consumed by the live MLSS in the clarifier itself. Thereafter, these MLSS will not have oxygen until the time they are returned to the aeration tank. It inhibits their metabolism on entering the aeration tank. Thus the return sludge shall not be really live. The depth also influences the hydraulic pattern. Higher depths may cause dead zones and shallow depths may cause short-circuiting between sewage released in the baffle zone and what overflows along the peripheral weirs. The data for clarifiers in STPs built in India and evaluated by NEERI are extracted in Appendix A.5.10.

It is seen that depths of primary clarifiers vary from 2.4 m to 4.2 m with detention times varying from 1.65 hours to 4 hours. In secondary clarifiers, the depths vary from 2.4 m to 4.2 m and detention times vary from 2.2 hours to 4.2 hours. Considering all these factors and the reported performance of these STPs, it requires an iterative approach. In the case of secondary clarifiers for extended aeration plants, deeper depths and longer detention times are not significant from return sludge point of view as the sludge is already mineralized when it leaves the aeration tank. In cases where marginally deeper depths and slightly longer detention times are to be considered for secondary clarifiers, the contact stabilization process is recommended to freshen up the sludge before returning it to the aeration tank.

5.7.4.2.5   Design Guidelines and Procedure

The design guidelines for both the primary and secondary clarifiers are given in Table 5.8.

Table 5.8 Design Parameters for Clarifiers

Type of Settling	Overflow rate, cum/sqm/day		Solid loading, kg/day/sqm		Side Water Depth, m	Weir loading, cum/m/day
	Average	Peak	Average	Peak	Average	Average
Primary Clarifiers						
Primary Settling only	25 - 30	50 - 60	Not applicable		≥ 2.5 - 3.5	125
Followed by secondary treatment	35 - 50	80 - 120	Not applicable		≥ 2.5 - 3.5	125
With excess sludge return	25 - 35	50 - 60	Not applicable		≥ 3.5 - 4.5	125
Secondary Clarifiers						
Secondary settling for activated sludge	15 - 35	40 - 50	70 - 140	210	≥ 3.0 - 3.5	185
Secondary settling for extended aeration	8 - 15	25 - 35	25 - 120	170	≥ 3.0 - 4.0	185

Note: Where the mechanized aerobic treatment is used after UASB reactor,
the settling tank design shall be based on conventional activated sludge process as above.

Source: CPHEEO, 1993 and as recommended in the present manual

The smaller values are for plants of less than 5 MLD. It is necessary to provide the clarifiers in at least two parallel units to have availability of one during repairs to the other. For bigger plants, more numbers are needed.

The procedure of sizing the clarifiers shall be as follows. In respect of all STPs where only BOD removal with or without biological nitrification is concerned, the hydraulic load from any return flows as sludge return, thickener supernatant return, sludge filtrate return are not taken into consideration and are deemed to be covered within the design criteria as in Table 5.8

In the case of biological nitrification-denitrification tanks, the flow due to internal return from the secondary clarifier overflow needs to be added to the average and peak flows for verifying the compliance to the design criteria. The recommended design procedures are furnished herein.

Design Procedure for Primary clarifiers:

1. Choose the average overflow rate in Table 5-8 and arrive at the surface area
2. Choose the peak overflow rate in Table 5-8 and arrive at the surface area
3. Choose the higher of the above two values and decide the diameter
4. Verify the weir overflow rate for compliance to Table 5-8
5. If the rate exceeds, verify with a double sided launder inside the clarifier
6. Even with this, if the rate exceeds, increase the diameter suitably.
7. Choose a compatible SWD.

Design Procedure for Secondary clarifiers:

1. Choose the average overflow rate in Table 5-8 and arrive at the surface area
2. Choose the peak overflow rate in Table 5-8 and arrive at the surface area
3. Choose the average solids loading rate in Table 5-8 and decide surface area
4. Choose the peak solids loading in Table 5-8 and arrive at the surface area
5. Choose the higher of the above four values and decide the diameter
6. Verify the weir overflow rate for compliance to Table 5-8
7. If the rate exceeds, verify with a double sided launder inside the clarifier
8. Even with this, if the rate exceeds, increase the diameter suitably.
9. Choose a compatible SWD.

An illustrative sizing of clarifiers in ASP is given in Appendix A.5.11

5.7.4.2.6   Sludge withdrawal from Clarifiers and Thickeners

There is some uncertainty in the issue of whether sludge from clarifiers and thickeners is to be drawn by first drawing it into a sludge sump and then withdrawing by a pump set.

The following clarity is now advocated.

5.7.4.2.6.1   Primary Clarifier for 10 MLD with 400 mg/l of SS & 60 % Removal

The SS removed per day will be 10×400×0.6 = 2400 kg/day

The solids concentration will be 2 %

The volume to be drawn will be 2400×(100/2)/1000 = 120 m^3/day

If drawn continuously for 24 hours, withdrawal has to be 120/24 = 5 m^3/hr

Using minimum specified pipe of dia 200 mm, area is (0.2)×(0.2)×0.785 = 0.03 m^2

Thus the velocity will be 5 / 3600 / 0.03 = 0.04 m/s

Clearly this cannot be permitted.

Suppose the sludge is drawn for 5 minutes every hour, then the flow will be revised as

Withdrawal has to be 120 / (24×5/60) = 60 m^3/hr

Using minimum specified pipe of dia 200 mm, area is (0.2)×(0.2)×0.785 = 0.03 m^2

Thus the velocity will be 60 / 3600 / 0.03 = 0.55 m/s

Obviously this is much better than withdrawal through a sludge sump.

5.7.4.2.6.2   Secondary Clarifier for 10 MLD of Sewage with 0.6 RAS

The volume to be drawn will be 10000×0.6 = 6000 m^3/day

If drawn continuously for 24 hours, withdrawal has to be 6000/24 = 250 m^3/hr

For a velocity of 1.5 m/s, area is 250/3600/1.5 = 0.046 m^2 resulting in a dia of 0.25 m

By using a sludge sump, the height has to be from invert of pipe to top of the clarifier

This height will be anywhere about 4.5 m.

For a minimum diameter of 2 m for man entry when needed, area is 3.14 m^2

Resulting volume will be 4.5×3.14 = 14.13 m^3.

HRT in the sump becomes 14.13/250 = 0.056 hrs or 3.4 minutes

Any sump has to be minimum 10 minutes of HRT

Hence the volume needed is 250×10/60 = 42 m^3

Required diameter becomes 3.5 m

It is a choice by the designer to decide on direct pumping or sump and pumping.

5.7.4.2.6.3   Choice of Sludge Withdrawal

a) Direct suction minimizes the complexities of sumps, valves, etc.

b) If the clarifier water level is just at ground level, it is necessary to construct a dry well and equip it with dry pit submersible pump sets of open impeller or centrifugal screw impeller directly coupled to the flange of the suction pipe after a valve on the upstream. The rpm shall be less than 960.

c) For RAS, if the designer prefers an intermediate sludge sump, necessary diffused air shall be let into such a sump. The pump sets can also be horizontal centrifugal foot mounted pump sets (in a separate dry pit) of motor not more than 960 rpm and equipped with VFD control or with submersible pump sets of speed not over 960 rpm or Archimedean screw pump in the sump itself which has to be rectangular to accommodate the screw.

Sludge can be removed either hydrostatically or mechanically from the sedimentation tanks. The sludge is withdrawn from the tank by hydrostatic pressure or by pumping. Manual cleaning has been largely given up in favour of mechanical cleaning in modern practice. Tanks are also provided with hopper bottoms for hydrostatic sludge removal.

Generally horizontal flow tanks are provided with rectangular hoppers and vertical tanks with circular or square types. Side slopes of the hoppers should be of the order of 1.2:1 to 2:1 preferably with values greater than 1.7:1 and 1.5:1 for pyramidal and conical hoppers respectively. The floor of the hoppers should not be wider than 0.6 m.

Mechanical sludge scraping is best suited for circular or square tanks and occasionally adopted in rectangular tanks. The scrapers or ploughs push the sludge along the tank bottom to sludge collecting channel or pocket from where it is either pumped directly or gravitated to a sludge sump for further disposal. In rectangular tanks, sludge hoppers are generally placed at the inlet end. However, they may be placed at mid-length in long tanks or at the outlet end in case of secondary settling tank of activated sludge plant. The sludge scraping mechanism may be of a moving bridge type of flight scrapers mounted on endless chain conveyors. The linear conveyor speed should not exceed 0.010 to 0.015 m/s.

In case of flight scrapers, where the maximum width of tanks is greater than twice the depth, multiple flight scrapers are placed side by side, in which case the width of tank could be increased up to a maximum 30 m. When multiple flight scrapers are used, the receiving sludge hoppers are designed as a trough with transverse collectors to convey the sludge to a single outlet pocket. A bottom slope of 1% is recommended for mechanical scraping of sludge.

The most common type of sludge scraping in circular clarifiers is a revolving sludge scraper mechanism with radial arms having ploughs or blades set at an angle just above floor level and rotating at 1 to 2 revolutions per hour. The ploughs push the sludge to a central hopper as the arms are rotated. Sludge from the central hopper is removed to a sludge sump by the side of the tank from where it is pumped or it is directly sucked and pumped out.

For small diameters of up to 9 m, the revolving bridge is spanned across the diameter of the tank. For bigger diameters, it is supported on the tank wall on one side and on a pillar at the centre of the tank. This pillar is a hollow construction and serves as the inlet to the clarifier. The drive motors for the sludge scraper arms can be either stationary at the centre of the walkway or movable in the case of traction drive and are mounted at one end of the rotating bridge. The interval between sludge removals should be less than 4 hours and never exceed 12 hours. Light flocculent sludge such as the activated sludge or mixture of activated sludge and primary sludge are scraped and shall be removed continuously from the tank to avoid septicity as the activated sludge are live organisms.

The peripheral speed of the scraper should be between 2.5 to 4 cm/sec. All rotary mechanisms are operated at a low speed of 1 to 2 revolutions per hour.

In the case of square tanks, the sludge scraper arms are provided with a pivoted extension with blades which will project out at the corners and retract to suit the diameter once it has crossed them as shown in Figure 5.36.
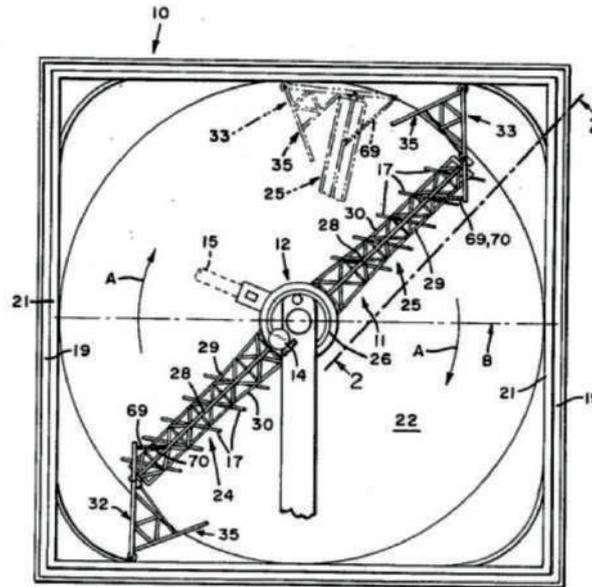
Source.US Patent Application Number 183106
Figure 5.36 Floor Plan of Square Shaped Clarifiers Sludge Scraper

The pantograph mechanism is numbered as 32 and 35 in Figure 5-36. These remove the sludge from the corners and push it towards the centre. The floor should be suitably finished. These types of clarifier construction is useful in the case of long rectangular aeration tanks where the longer sidewalls can be used to attach primary and secondary clarifiers on each of the shorter sides or on the longer side as abutting clarifiers using common wall construction for saving land area and for superstructure for covering all these for indoor air quality control in due course.

Where sludge is removed intermittently with intervals longer than four hours, provision for sludge storage in the hoppers of the tanks should be made. Sludge conveyor pipes should not be less than 200 mm in diameter. Hopper volumes should be excluded when calculating the effective sedimentation volume of the tank.

As the withdrawal from primary clarifiers is on an intermittent basis, the connecting pipe between the clarifier and the sludge sump outside the clarifier gets into choking problems besides the sludge in the sludge sump also getting anaerobic and malodorous due to storage. There are also installations where the sludge is withdrawn by direct suction.

It is recommended to adopt a sludge withdrawal every hour and of just the adequate volume to not induce the tunnel effect in the sludge zone of the clarifier. The pump impellers are to be preferred as positive displacement stator-rotor or screw centrifugal horizontal foot mounted type working at not over 960 rpm.

5.7.4.2.7 Inlets and Outlets

Performance of clarifiers is very much influenced by inlet and outlet devices. The inlet devices must distribute the flow evenly in the tank.

All inlets must be designed to keep down the entrance velocity to prevent formation of eddy or inertial currents in the tank to avoid short-circuiting. A stilling chamber is necessary ahead of inlets if the sewage is received under pressure from pumping mains.

The design should ensure least interference with the settling zone and promote ideal settling conditions. The outlet devices like the peripheral weirs must withdraw the sewage uniformly over the full length. Hence, the weir elevation must be kept the same all around the periphery.

The choice of inlet and outlet design depends on the geometry of sedimentation tank and the mode of entry and exit from the tank.

In horizontal flow rectangular tanks, inlets and outlets are placed opposite each other separated by the length of tank with the inlet perpendicular to the direction of flow.

In the design of inlets to rectangular tanks the following methods are used to distribute the flow uniformly across the tank.

a) Multiple pipe inlets with baffle boards of depth 0.45 to 0.6 m in front of the inlets, 0.6 to 0.9 m away from it, and with the top of baffle being 25 mm below water surface for the scum to pass over

b) Channel inlet with perforated baffle side wall between the tank and the channels, or

c) Inlet channel with submerged weirs followed by a baffle board inside the tank.

Scum baffles are provided ahead of outlet devices to prevent the escape of scum with the effluent.

In radial flow circular tanks, the usual practice is to provide a central inlet and a peripheral outlet. The central inlet pipe may be either a submerged horizontal pipe from wall to centre or an inverted siphon laid beneath the tank floor or a top entry pipe suspended from the bridge.

An inlet baffle is placed concentric to the pipe mouth, generally with a diameter of 10 to 20 % of the tank diameter and extending 1 to 2 m below water surface. Where the inlet pipe discharges into a central hollow pillar, the top of the pillar is flared to provide adequate number of inlet diffusion ports through which sewage enters the tank with an entry velocity of 0.10 to 0.25 m/s through the ports. The entry ports are submerged 0.3 to 0.6 m below the water surface.

The outlet is generally a peripheral weir discharging freely into a peripheral channel, the crest of the weir is provided with V-notches for uniform draw off at low flows or finished as bevelled masonry wears as in Figure 5.35. In all primary clarifiers, a peripheral scum baffle extending 0.20 to 0.30 m below the water surface is provided ahead of the effluent weir.

If the length of the peripheral weir is not adequate, a weir trough mounted on wall brackets near the periphery with adjustable overflow weir on both sides is provided to increase the length of weir.

5.7.4.2.8 Scum Removal

One distinct feature of primary clarifiers is the skimming device, which could be operated by the same drive mechanism as the sludge scraper arms at the bottom of the tank. It generally consists of a skimmer arm to which a scraper blade is attached and moved, partly submerged and partly projecting above the water surface, from the outlet end towards the inlet end in case of rectangular tanks or in a circular path in the case of circular tanks. The floating scum is thus collected at the forward end of the scraper blade and moved until it is tipped manually or automatically into a scum trough, which discharges the scum to a sump outside the tank from where it is removed for burial, burning or feeding to the digester.

A scum baffle at least 0.15 m above and extending to at least 0.30 m below the water level is provided along the periphery, ahead of outlet device, to prevent the escape of scum with effluent.

5.7.4.2.9 Types and Shapes

Circular tanks are more common than rectangular or square tanks. Up-flow tanks have been used for sewage sedimentation, but horizontal flow types are more popular. Rectangular tanks need less space than circular tanks and can be more economically designed in a large plant, where multiple units are to be constructed. They can form a more compact layout with the rectangular secondary treatment units such as aeration tanks in the activated sludge system. The diameters of circular tanks vary widely from 3 m to 60 m although the most common range is 12 m to 30 m. The diameters and depths could be chosen at the discretion of the designer in conformity with the manufactured sizes of scraper mechanisms in the country. The water depth shall be as per Table 5.8. Floors are sloped from periphery to centre at a rate of 7.5 % to 10%. The inlet to the tank is generally at the centre and outlet is a peripheral weir, the flow being radial and horizontal from centre to the periphery of the tank. Multiple units are arranged in pairs with feed from a central control chamber.

For rectangular tanks, maximum length and widths of 90 m and 30 m respectively with length to width ratios of 1.5 to 7.5 and length to depth ratios of 5 to 25 are recommended and depths shall be compatible with the sludge moving equipment manufacturer's requirements. Bottom slopes of 1% are normally adopted. Peak velocities shall not exceed 1.5 mph.

5.7.5 Performance

Primary clarifiers may be expected to accomplish 30% to 45% removal of BOD, (but shall be taken as maximum of 35% for design) and 60%-70% removal of SS, (but shall be taken as maximum of 60 % for design) depending on concentration and characteristics of solids in suspension. Secondary clarifiers, if considered independently, remove a very high percentage of flocculated solids, even more than 99%, particularly following an activated sludge unit where high mixed liquor suspended solids concentration is maintained in the aeration tank.

However, the efficiency of the biological treatment process is always defined in terms of the combined efficiency of the biological treatment units and its secondary clarifier with reference to the characteristics of the incoming sewage.

5.7.6   Chemical Aided Sedimentation

Chemical aided sedimentation of sewage is not normally recommended in the scope of biological treatment plants, unless it is warranted with reference to needs of compliance with quality of treated sewage, especially to control the residual phosphorous. Sometimes, when biological nitrification is aimed at and the required bicarbonate alkalinity is not inherent in the sewage, Sodium carbonate or bicarbonate will be necessary. In practice, it is analogous to chemical coagulation, flocculation and sedimentation in water treatment. The colloidal and finely dispersed solids which cannot be removed by plain primary sedimentation alone as they possess extremely low settling velocities and are aggregated into settleable particles by addition of chemicals in chemical-aided sedimentation. Commonly used chemicals are salts of lime, aluminium, ferric and ferrous in the form of powder or solutions, polyelectrolytes and polymers.

5.7.6.1   Unit Operations

The process consists of the three unit operations viz., proportioning and mixing of chemicals, flocculation and sedimentation.

5.7.6.1.1   Mixing

The required dose of chemical is weighed and fed to sewage by means of proportioning and feeding devices, ahead of the mixing unit. Mixing is accomplished in a rapid or flash mixing unit provided with paddles, propellers or by diffused air and having detention period of 0.5 to 3 minutes. The paddles of propellers are mounted on a vertical shaft and driven by a constant speed motor through reduction gears. The size and speed of the propeller is so selected as to give a propeller capacity of twice the maximum flow through the tank.

The shaft speed is generally between 100 to120 rpm and power needed is about 0.1 kW / MLD.

5.7.6.1.2   Flocculation

The principle of flocculation in sewage is similar to flocculation in water purification. The flocs that are formed after flash mixing with chemicals are made to coalesce into bigger sizes by either air flocculation or mechanical flocculation. Both diffused air and mechanical vertical draft tube are used for air flocculation. Revolving paddle type is the most common of the mechanical flocculators. The tanks are usually in duplicate with a detention period of 30-90 minutes depending upon results required and the type of sewage treated. However, the combination of chemical dosage and the flocculation period are first determined by laboratory jar test followed by bench scale testing in the field. The paddles are mounted either on a horizontal or vertical shaft. The peripheral speed of the paddles is kept in the range of 0.3 to 0.45 m/s. The flow-through velocity through the flocculator shall be in the range of 15 to 25 cm/sec to prevent sedimentation there itself. The drive motors can be either stationary or movable in the case of traction drive and are placed above the tank. In case of domestic sewage and certain industrial wastes, mechanical flocculation without addition of chemicals will reduce self-flocculation of the finely divided suspended solids and hence increase the efficiency of sedimentation.

5.7.6.1.3   Sedimentation

The flocculated sewage solids are settled out in a subsequent sedimentation tank. Refer Appendix 7.2 to 7.6 of CPHEEO Manual on Water Supply & Treatment 1999.

5.8   SEWAGE TREATMENT

Sewage Treatment detailed here will be on biological treatment technology only. It covers such of those technologies for which validated design guidelines are available in India over the past many decades. There are more recent technologies with each of them having their own design guidelines by the respective equipment vendors and for which obviously there are proprietary issues in procurement out of public funds. No doubt, unless these are tried out at some point in time, there is no way of inheriting these forever, but at the same time, the proprietary issue has to be got over. Hence, these technologies will be addressed later in this chapter under the title "Recent Technologies". Accordingly, the technologies to be considered in this chapter will be the Activated Sludge Processes, Attached Growth Systems, Treatment Methods Using Immobilization Carrier, Stabilization Ponds and Anaerobic Treatment. It is decided to phase out the stone media trickling filter technology considering the difficulties of upkeep of its rotary distributor, Psychoda flies nuisance and the recent light weight media which give much more surface area for unit volume of the media as compared to the stone media.

5.8.1   Activated Sludge Process

5.8.1.1 Introduction

Aerobic suspended growth systems are of two basic types, those which employ sludge recirculation, viz., conventional activated sludge process and its modifications and those which do not have sludge recycle, viz., aerated lagoons. In both cases sewage containing organic matter is aerated in an aeration basin in which micro-organisms metabolize the soluble and suspended organic matter. Part of the organic matter is synthesized into new cells and part is oxidized to carbon dioxide and water to derive energy. In activated sludge systems the new cells formed in the reaction are removed from the liquid stream in the form of a flocculent sludge in clarifiers. A part of this activated sludge is recycled to the aeration basin and the remaining form waste or excess sludge. In aerated lagoons the microbial mass leaves with the effluent stream or may settle down in areas of the aeration basin where mixing is not sufficient.

The suspended solids concentration in the aeration tank liquor, also called mixed liquor suspended solids (MLSS), is generally taken as an index of the mass of active micro-organisms in the aeration tank. However, the MLSS will contain not only active micro-organisms but also dead cells as well as inert organic matter derived from the raw sewage. The mixed liquor volatile suspended solids (MLVSS) value is also used and is preferable to MLSS as it eliminates the effect of inorganic matter. Aerobic and facultative bacteria are the predominant micro-organisms which carry out the above reactions of organic matter i.e. oxidation and synthesis. Their cellular mass contains about 12% Nitrogen and 2% Phosphorous. These nutrients should be present in sufficient quantity in the waste or they may be added, as required, for the reactions to proceed satisfactorily. A generally recommended ratio of BOD:N:P is 100:5:1. Domestic sewage is generally balanced with respect to these nutrients.

5.8.1.2   Activated Sludge Process Variables

An ASP essentially consists of the following: (i) Aeration tank containing microorganisms in suspension in which the reaction takes place, (ii) Activated sludge recirculation system, (iii) Excess sludge wasting and disposal facilities, (iv) Aeration systems to transfer oxygen and (v) Secondary sedimentation tank to separate and thicken activated sludge. These are schematically illustrated in Figure 5.6 (a) to (e). The main variables of the ASP are the loading rate, the mixing regime and the flow scheme.

5.8.1.3   Loading Rate

The loading rate expresses the rate at which the sewage is applied in the aeration tank. A loading parameter that has been developed empirically over the years is the hydraulic retention time (HRT), θ, d.

$$\theta = \frac{V}{Q}$$

(5.22)

Where,

       V : Volume of aeration tank, m^3, and

       Q : Sewage inflow, m^3/day

Another empirical loading parameter is volumetric organic loading which is defined as the BOD applied per unit volume of aeration tank, per day.

A rational loading parameter which has found wider acceptance and is preferred, is specific substrate utilization rate, U, per day which is defined as:

$$U = \frac{Q(S_0 - S)}{VX}$$

(5.23)

A similar loading parameter is mean cell residence time or sludge retention time (SRT), θc, day:

$$\theta_c = \frac{VX}{Q_w X_s}$$

(5.24)

where S_0 and S are influent and effluent organic matter concentrations respectively, conventionally measured as BOD_5, (g/m^3) X and Xs are MLSS concentration in aeration tank and waste activated sludge from secondary settling tank under flow, respectively, (g/m^3) and Qw - waste activated sludge rate, (m^3/d). Under steady state operation the mass of waste activated sludge is given by

$$Q_w X_s = YQ(S_0 - S) - k_d XV$$

(5.25)

where,

      Y :   Maximum yield coefficient (microbial mass synthesized/mass of substrate utilized)

      k_d :   Endogenous respiration rate constant, (d^{-1}).

From the earlier equations it is seen that

$$1/\theta_c = YU - k_d \qquad\qquad (5.26)$$

Since both Y and kd are constants for a given waste, it is, therefore, necessary to define either θ_c or U. Equation (5.26) is plotted in Figure 5.37 for typical values of Y = 0.5 and k_d = 0.06/d for municipal sewage.
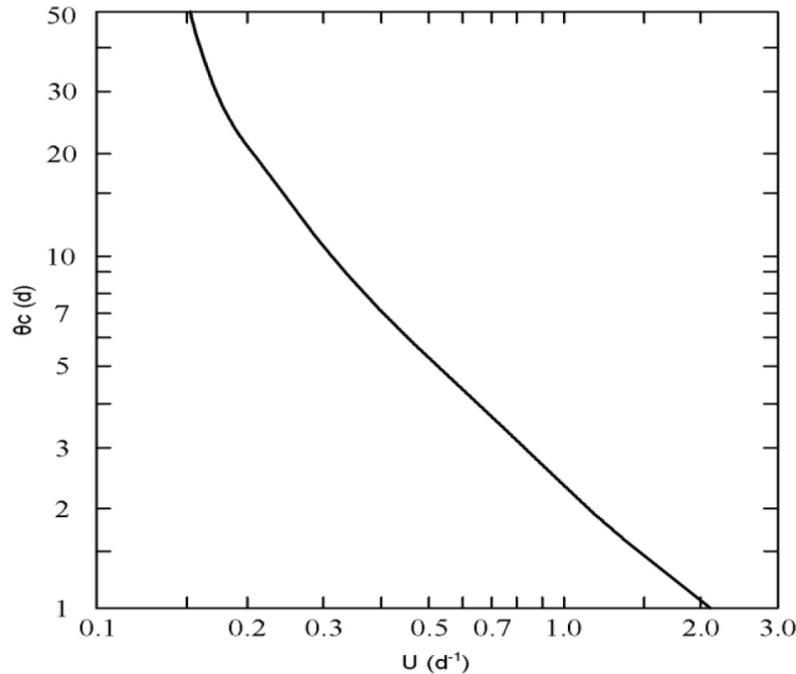
Figure 5.37 Relationship between SRT (θ_c) and specific substrate utilization rate
(U) for Y = 0.5 and k = 0.06d[-1]

If the value of S is small compared to S_0, which is often the case for activated sludge systems treating municipal sewage, U may also be expressed as Food applied to Microorganism ratio,

$$F/M = QS_0 / XV \qquad\qquad (5.27)$$

The θ_c value controls the effluent quality, settleability and drainability of biomass.

Other operational parameters which are affected by the choice of θ_c values are oxygen requirement and quantity of waste activated sludge.

Figure 5-38 (overleaf) gives θc value as a function of temperature for 90-95% reduction of BOD of municipal sewage.

5.8.1.4   Design Criteria

Typical values of loading parameters for various activated sludge modifications commonly used in India are furnished in Table 5.9 overleaf.
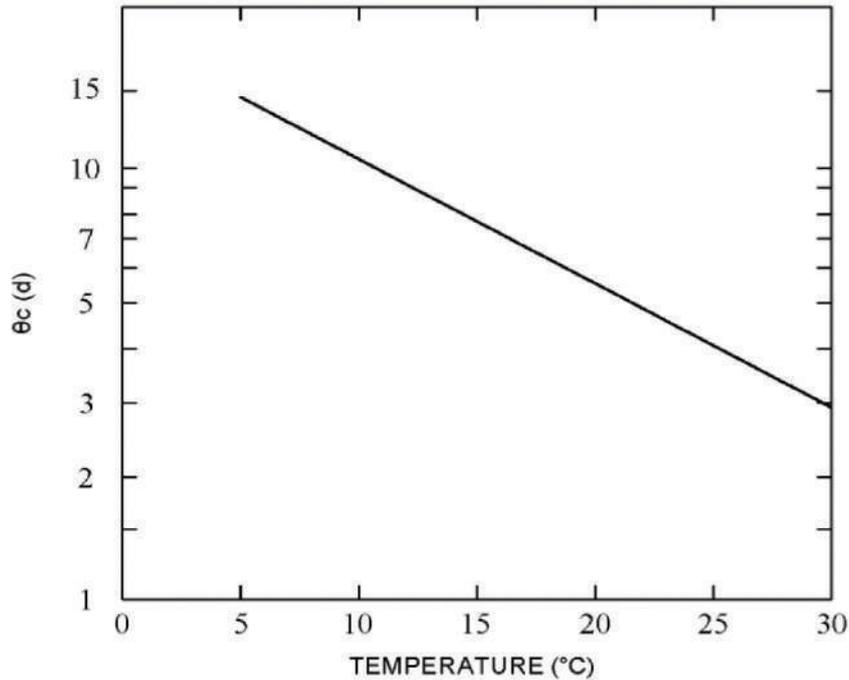
Source: CPHEEO, 1993

Figure 5.38 SRT as a function of aeration basin temperature for 90-95% BOD removal

Table 5.9 Characteristics and Design Parameters of Activated Sludge Systems for Sewage

Process Type	unit	Flow Regime		
		Conventional	Complete mix	Extended aeration
MLSS	mg/L	1500 to 3000	3000 to 4000	3000 to 5000
MLSS/MLVSS	ratio	0.8	0.8	0.6
F/M	day^{-1}	0.3 to 0.4	0.3 to 0.6	0.1 to 0.18
HRT	Hours	4 to 6	4 to 6	12 to 24
θc	days	5 to 8	5 to 8	10 to 26
Q_R/Q	ratio	0.25 to 0.5	0.25 to 0.8	0.25 to 1.0
BOD removal	%	85 to 92	85 to 92	95 to 98
kg O_2/kg BOD removed	ratio	0.8 to 1.0	0.8 to 1.0	1.0 to 1.2

Source: CPHEEO, 1993

5.8.1.5 Mixing Regime

The mixing regime employed in the aeration tank may be plug flow or completely mixed flow. Plug-flow implies that the sewage moves down progressively along the aeration tank essentially unmixed with the rest of the tank contents. Completely mixed flow involves rapid dispersal of the incoming sewage throughout the tank. In the plug flow system, the F/M and the oxygen demand will be highest at the inlet end of the aeration tank and will then progressively decrease. In the completely mixed system, the F/M and the oxygen demand will be uniform throughout the tank.

5.8.1.6   Flow Scheme

The flow scheme involves the pattern of sewage addition and sludge return to the aeration tank and the pattern of aeration. Sewage addition may be at a single point at the inlet end of the tank or it may be at several points along the aeration tank. The sludge return may be directly from the settling tank to the aeration tank or through a sludge reaeration tank. Aeration may be at a uniform rate or it may be varied from the head of the aeration tank to its end.

5.8.1.7   Conventional System and Modifications

The conventional system represents the early development of the activated sludge process. Over the years, several modifications to the conventional system have been developed to meet specific treatment objectives by modifying the process variables discussed earlier.

In step aeration, settled sewage is introduced at several points along the tank length, which produces uniform oxygen demand throughout. Tapered aeration attempts to supply air to match oxygen demand along the length of the tank. Contact stabilization provides for reaeration of return activated sludge from the final clarifier, which allows a smaller aeration or contact tank. While the conventional system maintains a plug flow hydraulic regime, completely mixed process aims at instantaneous mixing of the influent waste and return sludge with the entire contents of the aeration tank. The extended aeration process employs low organic loading, long aeration time, high MLSS concentration and low F/M. Because of long detention in the aeration tank/oxidation ditch, the MLSS undergo considerable endogenous respiration and get well stabilized. The excess sludge does not require separate digestion and it can be dried directly on sand beds or mechanically dewatered. In addition, the excess sludge production is minimal. The conventional system and the last two modifications named above have found wider acceptance. These are described below in detail.

5.8.1.7.1   Conventional System

The Conventional system is always preceded by primary settling. The plant itself consists of a primary clarifier, an aeration tank, a secondary clarifier, a sludge return line and an excess sludge waste line leading to a digester. The BOD removal in the process is about 85% to 92%. The plant employs a plug flow regime, which is achieved by a long and narrow configuration of the aeration tank with length equal to 5 or more times the width. The sewage and mixed liquor enter at the head of the tank and is withdrawn at its end. Because of the plug flow regime, the oxygen demand at the head of the aeration tank is high and then tapers down. However, air is supplied in the process at a uniform rate along the length of the tank. This leads to either oxygen deficiency in the initial zone or wasteful application of air in the subsequent reaches. Another limitation of the plug flow regime is that there is a lack of operational stability at times of excessive variation in rate of inflow and in influent strength. For historical reasons, the conventional system is the most widely used type of the activated sludge process. Plants up to 300 MLD capacities have been built in India.

5.8.1.7.2   Completely Mixed

The complete mix activated sludge plant employs a completely mixed flow regime. In a circular or square tank, complete mixing is achieved by mechanical aerators with adequate mixing.

The completely mixed plant has the capacity to hold a high MLSS level in the aeration tank enabling the aeration tank volume to be reduced. The plant has increased operational stability at shock loadings and increased capacity to treat toxic biodegradable wastes like phenols.

5.8.1.7.3   Plug Flow

This occurs in tanks of a longish shape in plan when used with surface aerators and almost all diffused aeration tanks.

5.8.1.7.4   Extended Aeration

The flow scheme of the extended aeration process and its mixing regime are similar to that of the completely mixed process except that primary settling is omitted. The process employs low organic loading, long aeration time, high MLSS concentration and low F/M. The BOD removal efficiency is high. Because of long detention in the aeration tank, the mixed liquor solids undergo considerable endogenous respiration and remain well stabilized. The excess sludge does not require separate digestion and can he dried directly on sand beds. Furthermore, the excess sludge production is minimal. The oxygen requirement for the process is higher and the running costs are correspondingly high. However, operation is rendered simple due to the elimination of primary settling and separate sludge digestion. The method is, therefore, well suited specially for small and medium size communities and zones of a larger city. In small plants intermittent operation of extended aeration systems may be adopted, intermittent aeration cycles are: (i) closing of inlet and aerating the sewage, (ii) stopping aeration and allowing the contents to settle and (iii) letting in fresh sewage which displaces an equal quantity of clarified effluent. Sludge is wasted from the mixed liquor. To handle continuous flows a number of units may be operated in parallel.

The oxidation ditch is one form of an extended aeration system having certain special features like an endless ditch for the aeration tank and a rotor for the aeration mechanism. The ditch consists of a long continuous channel usually oval in plan. The channel may be earthen with lined sloping sides and lined floor or it may be built in concrete or brick with vertical walls. The sewage is aerated by a surface rotor placed across the channel. The rotor not only aerates the sewage but also imparts a horizontal velocity to the mixed liquor preventing the biological sludge from settling out.

5.8.1.7.5   Design Consideration

The items for consideration in the design of activated sludge plant are aeration tank capacity and dimensions, aeration facilities, secondary sludge settling and recycle and excess sludge wasting.

5.8.1.7.5.1   Aeration Tank

Equations (5.24) to (5.26) can be combined to yield

$$VX = YQ\theta_c (S_0 - S)/(1 + K_d\theta_c) \tag{5.28}$$

The volume of the aeration tank is calculated for the selected, value of θc by assuming a suitable value of MLSS concentration, X, in Equation (5.28).

Alternatively the tank capacity may be designed from F/M and MLSS concentration according to Equation (5.27). The F/M and MLSS levels generally employed in different types of commonly used activated sludge systems are given in Table 5-9 along with their corresponding BOD removal efficiencies.

It is seen that economy in reactor volume can be achieved by assuming a large value for X. However, it is seldom taken to be more than 5,000 g/m^3. A common range is between 1,000 and 4,000 g/m^3. Considerations that govern the upper limit are

a) Initial and running cost of sludge recirculation system to maintain a high value of MLSS,

b) Limitations of oxygen transfer equipment to supply oxygen at required rate in a small reactor volume,

c) Increased solids loading on secondary clarifier which may necessitate a larger surface area to meet limiting solid flux, design criteria for the tank and minimum HRT for the aeration tank for stable operation under hydraulic surges.

Except in the case of extended aeration plants and completely mixed plants, the aeration tanks are designed as long narrow channels. This configuration is achieved by the provision of round-the-end baffles in small plants when only one or two tank units are proposed and by construction as long and narrow rectangular tanks with common intermediate walls in large plants when several units are proposed. In extended aeration plants other than oxidation ditches and in complete mix plants the tank shape may be circular or square when the plant, capacity is small, or rectangular with several side inlets and equal number of side outlets, when the plant capacity is large.

The width and depth of the aeration channel depends on the type of aeration equipment employed. The depth controls the aeration efficiency and usually ranges from 3 m to 4.5 m for surface aerators, the deeper depth being justified by use of hopper bottomed tank square cells and draft tubes. In the case of diffused aeration, the delivery pressure at the compressor plays a crucial part in that, where this exceeds about 6.5 m depth water cooled compressors would be needed and this shall be duly considered. If the capacities are over 70 MLD then duplicate units are preferred.

The width controls the mixing and is usually 5 to 10 m. The width-depth ratio should be adjusted to be 1.2 to 2.2. The length should not be less than 30 m or not ordinarily longer than 100 m in a single section length before doubling back. The horizontal velocity should be around 1.5 m/min. Excessive width may lead to settlement of solids in the tank. Triangular baffles and fillets are used to eliminate dead spots and induce spiral flow in the tanks. The tank free-board is generally kept between 0.3 m and 0.5 m.

Due consideration must be given in the design of aeration tanks for the requirement of emptying them for maintenance and repair of the aeration equipment. Intermediate walls should be designed for empty conditions on each side.

The method of dewatering should be considered in the design and provided for during construction.

The inlet and outlet channels of the aeration tank should be designed for empty conditions on either side. The method of dewatering should be considered in the design and provided for during construction. The unit dewatering can be as per Section 5.3.13 already detailed in this manual.

The inlet and outlet channels of the aeration tanks should be designed to maintain a minimum velocity of 0.3 m/s to avoid deposition of solids. The channels or conduits and their appurtenances should be sized to carry the maximum hydraulic load to the remaining aeration tank units when any one unit is out of operation.

The inlet should provide for free fall into aeration tank when more than one tank unit or more than one inlet is proposed. The free fall will enable positive control of the flows through the different inlets. Outlets usually consist of free fall weirs. The weir length should be sufficient to maintain a reasonably constant water level in the tank. When multiple inlets are involved, they should be provided with valves, gates or stop planks to enable the regulation of flow through each inlet.

5.8.1.7.5.2   Oxygen Requirements

Oxygen is required in the activated sludge process for the oxidation of a part of the influent organic matter and for the endogenous respiration of the micro-organisms in the system. The total oxygen requirement of the process may be formulated as follows:

$$O_2 \text{ required (g/d)} = (Q (S_o-S)/f) - 1.42 \, \Delta x \qquad\qquad (5.29)$$

where,

f :   Ratio of BOD to ultimate BOD

1.42 :   Oxygen demand of biomass, g/g

Δ_X :   Biological sludge produced per day.

$\Delta_X =$ $Q \times Y_{obs} \times (S_o\text{-}S)$

$Y_{obs} =$ $Y/(1+K_d \times \theta_c)$

Where   Y is 0.5 and K_d is 0.06

The formula does not allow for nitrification but allows only for carbonaceous BOD removal. The extra theoretical oxygen requirement for nitrification is 4.56 Kg O_2/per kg NH_3 -N oxidized to NO_3 – N. The total oxygen requirements per kg BOD removed for different activated sludge processes are given in Table 5.9. The amount of oxygen required for a particular process will increase within the range shown in the Table 5.9 as the F/M value decreases. Appendix A.5.12 presents an illustrative design of conventional ASP aeration.

5.8.1.7.5.3   Aeration Facilities

The aeration facilities of the activated sludge plant are designed to provide the calculated oxygen demand of the sewage against a specific level of dissolved oxygen in the sewage. The aeration devices, apart from supplying the required oxygen demand shall also provide adequate mixing or agitation in order that the entire mixed liquor suspended solids present in the aeration tank will be available for the biological activity.

The recommended dissolved oxygen concentration in the aeration tank is in the range 0.5 to 1 mg/l for conventional activated sludge plants and in the range 1 to 2 mg/l for extended aeration type activated sludge plants and above 2 mg/l when nitrification is required in the ASP.

Aerators are rated based on the amount of oxygen they can transfer to tap water under standard conditions of 20°C, 760 mm Hg barometric pressure and zero DO. The oxygen transfer capacity under field conditions can be calculated from the standard oxygen transfer capacity by the formula:

$$N = \frac{N_s (C_s - C_L) \times 1.024^{(T-20)} \alpha}{9.17} \tag{5.30}$$

where,

N :   Oxygen transferred under field conditions, kg O_2/kWh

N_s :   Oxygen transfer capacity under standard conditions, kg O_2/kWh

C_s :   Dissolved oxygen saturation for sewage at operating temperature, mg/l

C_L :   Operation DO level in aeration tank usually 1 to 2 mg/1

T :   Temperature, °C

α :   Correction factor for oxygen transfer for sewage,

The value of C_s is calculated by arriving at the dissolved oxygen saturation value for tap water at the operating temperature and altitude as in Table 5.10 and Table 5.11 (overleaf) and then multiply it by a factor (β) which is usually 0.95 for domestic sewage and with TDS in the normal range of 1,200 to 1,500 mg/l.

The value of α requires a detailed understanding. This represents the ratio of the oxygen uptake rate, known as K_{La}, of the given sewage to that of clean tap water at 20 degree Celsius. In simple terms, it is the rate at which oxygen can be dissolved into water and sewage. The K_{La} for water is almost constant. It will however vary in sewage because of the constituents like organic matter, chemicals, biological organisms, detergents, etc, which interfere with the oxygen transfer. Also, the sewage quality itself varies between ULBs depending on water supply rates. Thus the K_{La} value for sewage will always be less than one. The importance of K_{La} as related to the α-value in STP is seen from equation 5.30.

The oxygen requirement at the standard condition denoted as N_s decides the kw of aeration equipment. This value of N_s is inversely proportional to the α-value. Thus, higher value of α-means lesser the kw of aerators and compressors and the entire aeration system. For example, if α-value is 0.8 in one case and 0.4 in another case, the cost of the entire aeration system using the value of 0.4 will be 200% higher. Thus, technically, it becomes highly debatable when specifying this α-value in DPR and in contracts. The second edition of the manual specifies this value as 0.8 to 0.85. This is also corroborated by the publication of Sundaramoorthy & Sundaresan (1972) which evaluated the α- value for the mixed liquor of a conventional ASP at Chennai as 0.847 to 0.854 for a surface aerator system. Many recent tender documents specify it as close to 0.6 in diffused aeration (BWSSB- Contract s1c for 60 MLD average flow BOD removal and Nitrification-denitrification). Even conceding that fine bubble aeration systems are more efficient in dissolving oxygen, Metcalf & Eddy (2003) cites the study by Hwand and Stenstrom (1985) reporting the α value as 0.4 to 0.9 for fine bubble diffuser system. Thus, there is considerable uncertainty in specifying α-value.

Table 5.10 DO Saturation vs. Temperature in Celsius in Tap Water at Mean Sea Level

The relationship between temperature and oxygen solubility	
Temperature(degree C)	Oxygen solubility (mg/l)
0	14.6
5	12.8
10	11.3
15	10.2
20	9.2
25	8.6
30	7.5
35	6.9
40	6.4
100 (boiling)	0.0

Table 5.11 DO Correction Factor for Altitudes

Altitude(meters)	Factor	Altitude(meters)	Factor
0	1	1067	0.88
152	0.98	1219	0.86
305	0.96	1372	0.84
457	0.95	1524	0.82
610	0.93	1676	0.81
762	0.91	1829	0.80
914	0.89		

At the same time, it should be recognized that the compressor capacity needed for ensuring adequate mixing energy is also important. In actual design, the power requirements are calculated separately for aeration & mixing and the higher of the two is chosen. Mostly, the power required for mixing is always higher.

Considering all these, it is considered prudent to opt for a α value of 0.6 for calculating the oxygen and hence the air requirements. If possible this value can also be got tested in the case of upgrading existing STPs

The oxygen transfer capacities of surface, fine and coarse diffused air systems under standard conditions lie between 1.2 to 2.4, 1.2 to 2 and 0.6 to 1.2 kg O_2/kWh respectively. However, it is necessary to secure the test certificates for the same from the diffused air system vendor before deciding on the tendered offers.

5.8.1.7.5.4   Diffused Aeration

Diffused aeration involves the introduction of compressed air into the sewage through submerged diffusers of fine bubble or coarse bubble type. In the former, compressed air is released at or near the bottom of the aeration tank through porous tubes or plates made of aluminium oxide or silicon oxide grains cemented together in a ceramic matrix. Troubles due to clogging from the inside can be reduced by providing air filters and those due to clogging from outside can be avoided by providing adequate air pressure below the diffusers at all times. In spite of such precautions, fine bubble diffusers will require periodical cleaning. Air supplied to porous diffusers should not contain more than 0.02 mg of dust per cum of air. Coarse bubble aerators have lower aeration efficiency than fine bubble aerators, but are cheaper in capital cost and are less liable to clogging and do not require filtration of air. In longish channel type aeration tanks, air diffusers are generally placed along one side of the aeration tank, helping to set up a spiral flow in the tank which improves mixing and prevents the solids from settling. They are located 0.3 m to 0.6 m above the tank floor to aid in tank cleaning and reduce clogging during shutdown. The air volume calculated based on the α-value shall be further adjusted as follows.

a) Percentage of oxygen in the air at the STP location

b) Weight of the air at the STP location

c) Efficiency of the diffusers in transferring the air at the given liquid depth of the aeration tank at the rate of 4 % to 5 % per metre depth.

d) Fouling factor of diffusers at the rate of 4 % to 5 % per year over its life span

e) An overall factor of safety of 10 % to take care of contingencies.

The compressor shall be provided for the above duty and it shall have a VFD so that the actual air requirement and hence the actually required kW alone can be operated to maintain the required residual oxygen in aeration.

The agitator-sparger is a mechanical aerator system involving the release of compressed air at the bottom of the aeration tank in large bubbles and the breaking up of the bubbles into fine bubbles by submerged turbine rotors located above the air outlets are also used.

5.8.1.7.5.5   Surface Aerators

Surface aerators are available in both fixed and floating types. Some of their advantages are higher oxygen transfer capacity, absence of air piping and air filter and simplicity of operation and maintenance. Surface aerators generally consist of large diameter impeller plates revolving on vertical shaft at the surface of the liquid with or without draft tubes. A hydraulic jump is created by the impellers at the surface causing air entrapment in the sewage. The impellers also induce mixing. The speed of rotation of the impellers is usually 70-100 rpm for geared motor systems. The aeration rotors for small oxidation ditches are generally of cage type but may also be of the angle iron type. Particular attention must be paid to the design of shaft length, bearings and alignment. Vertical shaft aerators are easier to maintain and are used with deeper ditches.

5.8.1.7.5.6 Mixing Requirements

The aeration equipment has also to provide adequate mixing in the aeration tank to keep the solids in suspension. The air requirements shall be calculated both for summer and winter as well as mixing power and the higher duty installed. Mixing considerations require that the minimum power input in activated sludge aeration tanks where MLSS is of the order of 4000-5000 mg/l, should not be less than 15-26 W/m^3 of tank volume. The power input of aerators derived from oxygenation considerations should be checked to satisfy the mixing requirements and increased where required. In the case of diffused aeration, the air volume for mixing shall be not less than 1.8-2.7 m^3/hr/m^2 of floor area (US EPA, 625/8-85/0100, p 38). The delivery head shall be as per the chosen liquid depth and friction losses. The surface area of the diffusers shall not be less than 6% of the floor area of the aeration tank. In the case of tubular diffusers, the centre to centre spacing shall be preferably restricted to not over 30 cm and where unavoidable, the interspaces shall be provided with pre-cast RCC ridges so that the MLSS if it settles down will slide to the diffusers and will be automatically pushed up into the aeration tank by the buoyancy. The loss of aeration tank volume by these ridge blocks and blocks for supporting the diffuser headers shall be compensated in deciding the liquid height of the aeration tank.

5.8.1.7.5.7 Measuring Devices

Devices should be installed for indicating flow rates of raw sewage or primary effluent, return sludge and air to each aeration tank. For plants designed for sewage flow of 10 MLD or more, integrating flow recorders should be used.

5.8.1.7.5.8 Secondary Settling

Secondary settling assumes considerable importance in the activated sludge process as the efficient separation of the biological sludge is necessary not only for ensuring final effluent quality but also for return of adequate sludge to maintain the MLSS level in the aeration tank. The secondary settling tank of the activated sludge process is particularly sensitive to fluctuations in flow rate and on this account it is recommended that the units be designed not only for average overflow rate but also for peak overflow rates. The high concentration of suspended solids in the effluent requires that the solids loading rate should also be considered. The recommended overflow rates and solids loading rates for secondary clarifiers of activated sludge have been given in Table 5.8.

5.8.1.7.5.9 Sludge Recycle

The MLSS concentration in the aeration tank is controlled by the sludge recirculation rate and the sludge settle ability and thickening in the secondary sedimentation tank.

$$\frac{Q_R}{Q} = \frac{X}{X_s - X}$$

(5.31)

where,

Q_R : Sludge recirculation rate, m^3/d.

The sludge settleability is determined by sludge volume index (SVI) defined as volume occupied in ml by one gram of solids in the mixed liquor after settling for 30 min and is determined experimentally. If it is assumed that sedimentation of suspended solids in the laboratory is similar to that in sedimentation tank, then $X_s = 10^6/\text{SVI}$. Values of SVI between 100 and 150 ml/g indicate good settling of suspended solids and this can be achieved for values suggested in Figure 5.38. The X_s value may not be taken more than 10,000 g/cum unless separate thickeners are provided to concentrate the settled solids or secondary sedimentation tank is designed to yield a higher value. Using the above value for X_s and 5000 mg/l for X in Equation (5.31), the sludge recirculation ratio comes out to be 1.0. The return sludge is always to be pumped and the recirculation ratio should be limited to the values suggested in Table 5.9.

As stated above, the recirculation ratio computation depends on the concentration of the sludge in the underflow of the clarifier and this in turn can be attributed to the SVI as mentioned. The SVI is a plant control parameter and cannot be assumed as a design parameter. Thus, the concentration of the sludge in the underflow of the clarifier is again not possible to pre-fix in design. Normally well operated clarifiers can be expected to concentrate the MLSS of mixed liquor by about 3 times. Thus, the thumb rule recirculation ratio can also be expressed as 1/(3-1) = 0.5. However, the thumb rule indicates a value of 0.25 to 0.8 in Table 5-9. Moreover, there has to be flexibility in the field to vary the recirculation ratio nearer to the higher limit to reach adequate flows and hence maintain velocities in piping through the plant when the influent sewage volume is very much less. Thus, it is recommended that irrespective of the designer's choice, the recirculation pump set shall be designed to deliver the higher volume but in actual practice the pumpage can be controlled to the bare minimum through a VFD control.

5.8.1.7.5.10   Excess Sludge Wasting

The sludge generated in the aeration tank has to be wasted to maintain a steady level of MLSS in the system. The excess sludge quantity will increase with increasing F/M and decrease with increasing temperature. The excess sludge generated under steady state operation may be estimated from Equation (5.24) and (5.25).

In the case of domestic sewage, the excess sludge to be wasted will be about 0.35-0.5 kg/kg BOD_5 removed for the conventional system and about 0.25-0.35 kg/kg BOD_5 removed in the case of extended aeration plants having no primary settling.

The volume of sludge to be wasted will depend on the suspended solids concentration in the waste stream.

Excess sludge may be wasted either from the sludge return line or directly from the aeration tank as mixed liquor. The latter procedure is to be preferred as the concentration of suspended solids will then be somewhat steady in the waste stream providing better control on biomass wasted.

The waste sludge is either discharged into the primary settling tank or thickened in a sludge thickening unit and digested directly, In extended aeration plants, the excess sludge is taken to sludge drying beds or mechanical dewatering directly and the sludge filtrate discharged into the effluent stream.

Excess sludge = (A/(0.6 to 0.8)) + B

A is calculated by the following equation and 0.6 to be used for extended aeration and 0.8 is used for conventional activated sludge.

$A = Q \times Y_{obs} (S_0 - S)$

$Y_{obs} = Y/(1 + K_d \times \theta_c)$

Where Y is 0.5

K_d is 0.06

B = Q×inert TSS removal

Inert TSS = Influent TSS – Influent VSS

TSS removal in primary settling tank is 60%.

Inert SS removal in primary settling tank is 60%.

VSS removal in primary settling tank is 60%.

θ_c is from Figure 5-38 for the lowest operating temperature.

Excess sludge in kg/day = Y_{obs} × BOD inlet × Flow MLD

Calculate excess sludge kg/day from the thumb rule in Section 5.8.1.7.5.10

Adopt the higher value

Excess sludge volume m^3/day = (Excess wasted kg/day) × 1000/MLSS in clarifier underflow

MLSS in clarifier underflow is to be assumed based on the SWD and is usually 3 times the MLSS.

5.8.1.7.6   Nitrification

Activated sludge plants are ordinarily designed for the removal of only carbonaceous BOD. However, there may be incidental nitrification in the process. Nitrification will consume part of the oxygen supplied to the system and reduce the DO level in the aeration tank. Nitrification will also lead to subsequent denitrification in the secondary clarifier causing a rising sludge problem also called blanket rising. Nitrification is aided by low F/M and long aeration time. It may be pronounced in extended aeration plants especially in hot weather. At the other extreme in the contact stabilization process and in the modified aeration plant, there may be little or no nitrification.

Nitrification though generally not desired may be required in specific cases, e.g. when ammonia has to be eliminated from the effluent in the interest of pisciculture or when nitrification cum denitrification is proposed for elimination of nitrogenous matter from the effluent for control of eutrophication. In such cases, plug flow systems have been developed for efficient removal of both carbon and nitrogen, Alternatively a two stage system may be designed with carbonaceous BOD removal in the first stage and nitrification in the second stage by ensuring adequate organic matter is still left behind at the end of the first stage to serve as the energy source for the nitrifying organisms in the second stage.

Nitrification requires bicarbonate alkalinity in the ratio of seven times that of the ammonia to be nitrified and if the available alkalinity is inadequate, the addition of Sodium carbonate or Bicarbonate is needed before the aeration tank.

5.8.1.7.7   Denitrification

In general, this is achieved as an integrated nitrification-denitrification process as a variation of the typical activated sludge process. The principle is shown in Figure 5.39 and the flow scheme is shown in Figure 5.40.
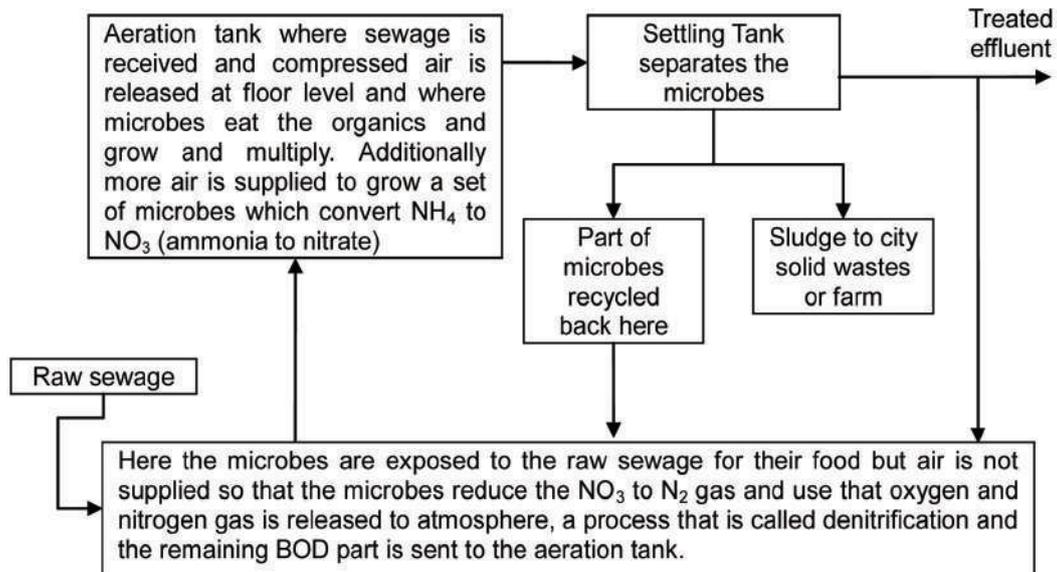
Figure 5.39 Schematic of biological nitrification-denitrification in activated sludge process
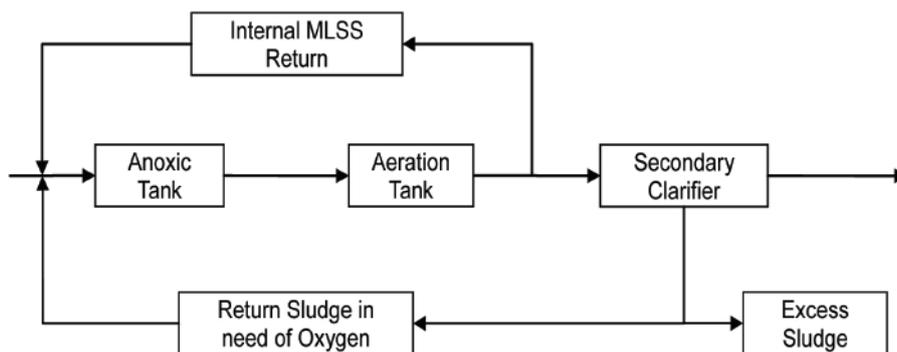
Figure 5.40 Flow routing in Activated Sludge Biological Nitrification-Denitrification process

5.8.1.7.8   Phosphorous Removal

The consciousness to restrict the phosphorous in the treated sewage before discharge into the environment to curtail eutrophication is being recognized. The phosphorous can be removed by a process called as the luxury uptake. There are at least six different variations of these processes which have all been developed in advanced countries and every situation will need a separate evaluation and validation.

An alternative process is to introduce a chemical precipitation either in the secondary clarifier or as a separate tertiary stage where phosphorous is precipitated by coagulating with Ferric or Aluminium salts. There is also another technology of high Lime followed by acidification or carbonation whereby in addition to phosphorous removal, colour, heavy metals, fluorides, silica and magnesium can also be simultaneously removed. It is necessary to conduct lab studies to establish the efficiency and the type of chemicals.

5.8.1.7.9   Aerated Lagoons

Aerated lagoons are generally provided in the form of simple earthen basins with inlet at one end and outlet at the other to enable the sewage to flow through while aeration is usually provided by mechanical means to stabilize the organic matter. The major difference between activated sludge systems and aerated lagoons is that in the latter, settling tanks and sludge recirculation are absent. Aerated lagoons are of two principal types depending on how the microbial mass of solids in the system is handled. Facultative Aerated Lagoons are those in which some solids may leave with the effluent stream and some settle down in the lagoon since aeration power input is just enough for oxygenation and not for keeping all solids in suspension. As the lower part of such lagoons may be anoxic or anaerobic while the upper layers are aerobic, the term facultative is used. Appendix A.5.13 presents an illustrative design of facultative aerated lagoon.

Aerobic Lagoons, on the other hand, are fully aerobic from top to bottom as the aeration power input is sufficiently high to keep all the solids in suspension besides meeting the oxygenation needs of the system. No settlement occurs in such lagoons and under equilibrium conditions the new (microbial) solids produced in the system equal the solids leaving the system. Thus, the solids concentration in the effluent is relatively high and some further treatment is generally provided after such lagoons. If the effluent is settled and the sludge recycled, the aerobic lagoon, in fact, becomes an activated sludge or extended aeration type lagoon. A few typical characteristics of the above types of lagoons are given in Table 5.12.

Table 5.12 Some Characteristics of Aerated Lagoons

No.	Characteristics	Facultative Aerated Lagoons	Fully Aerobic	Extended Aeration System (for comparison)
1.	Detention time, days	3 - 5	2 - 3	0.5 - 1.0
2.	Depth, m	2.5 – 5.0	2.5 – 4.0	2.5 - 4.0
3.	Land required, m²/person	0.15 - 0.30	0.10 - 0.20	
4.	BOO removal efficiency %	80 - 90	50 - 60	95 - 98
5.	Overall BOD removal rate, K (A)	0.6 – 0.8	1-1.5	20 - 30
6.	Suspended solids in lagoon, mg/l	40 - 150	150 - 350	3,000 - 5,000
7.	VSS/SS	0.6	0.8	0. 6
8.	Desirable power level (B)	0.75	2.75 - 6.0	15 - 18
9	Power requirement, kWh/person/year	12 - 15	12 - 14	16 - 20

Source: CPHEEO,1993

(A) Per day at 20 degree C for soluble BOD only; (B)-in watts per cum of lagoon volume

Facultative type aerated lagoons have been more commonly used the world over because of their simplicity in operation and minimum need of machinery. They are often referred to simply as 'aerated lagoons'. Their original use came as a means of upgrading overloaded oxidation ponds in some countries without adding to the land requirement. In fact, much less land is required compared to oxidation ponds.

In earlier times the design of aerated lagoons was often done using simple thumb-rules of detention time and power per capita. However, over the years it has come to be recognized that lagoons being large bodies of water are subject to seasonal temperature effects and flow mixing conditions. Flow conditions in aerated lagoons are neither ideal complete-mixing nor ideal plug-flow in nature. They are dependent on lagoon geometry and are better described by dispersed flow models of the type given by Wehner and Wilhem for first-order kinetics and hence the design procedure given below takes treatability of the waste, temperature and mixing conditions into account. Fully aerobic lagoons always have a complete-mixing regime and a slightly different mode of design is followed. However, as aerobic lagoons have not yet been built in India (except one case) further discussion is limited to facultative aerated lagoons only.

5.8.1.7.10   Design Variables

For facultative aerated lagoons, the dispersed flow model just referred to gives the relation between influent and effluent substrate concentrations, S_0 and S, respectively and other variables such as the nature of the waste, the detention time and the mixing conditions, as shown in the following equation.

$$\frac{S}{S_0} = \frac{4ae^{1/2d}}{(1+a)^2 e^{a/2d} - (1-a)^2 e^{-a/2d}}$$

in which the term $a = \sqrt{1 + 4K\theta d}$

d : dispersion number (dimensionless)

$= D/UL = D\theta/L^2$

(5.32)

where,

D :   Axial dispersion coefficient (length^2/time)

L :   Length of axial travel path

θ :   Theoretical detention time. (Volume/Flow rate)

U :   Velocity of flow through lagoon (length/time)

K :   Substrate removal rate in lagoon (time^{-1})

S_0 & S : Initial and final substrate concentrations (mass/volume)

A graphical solution of the above equation is shown in Figure 5.41 (overleaf) from which it is seen that prior knowledge of the substrate removal rate K as well as of the mixing condition likely to prevail in a lagoon is necessary to determine the efficiency of BOD removal at selected detention time. This is discussed further.
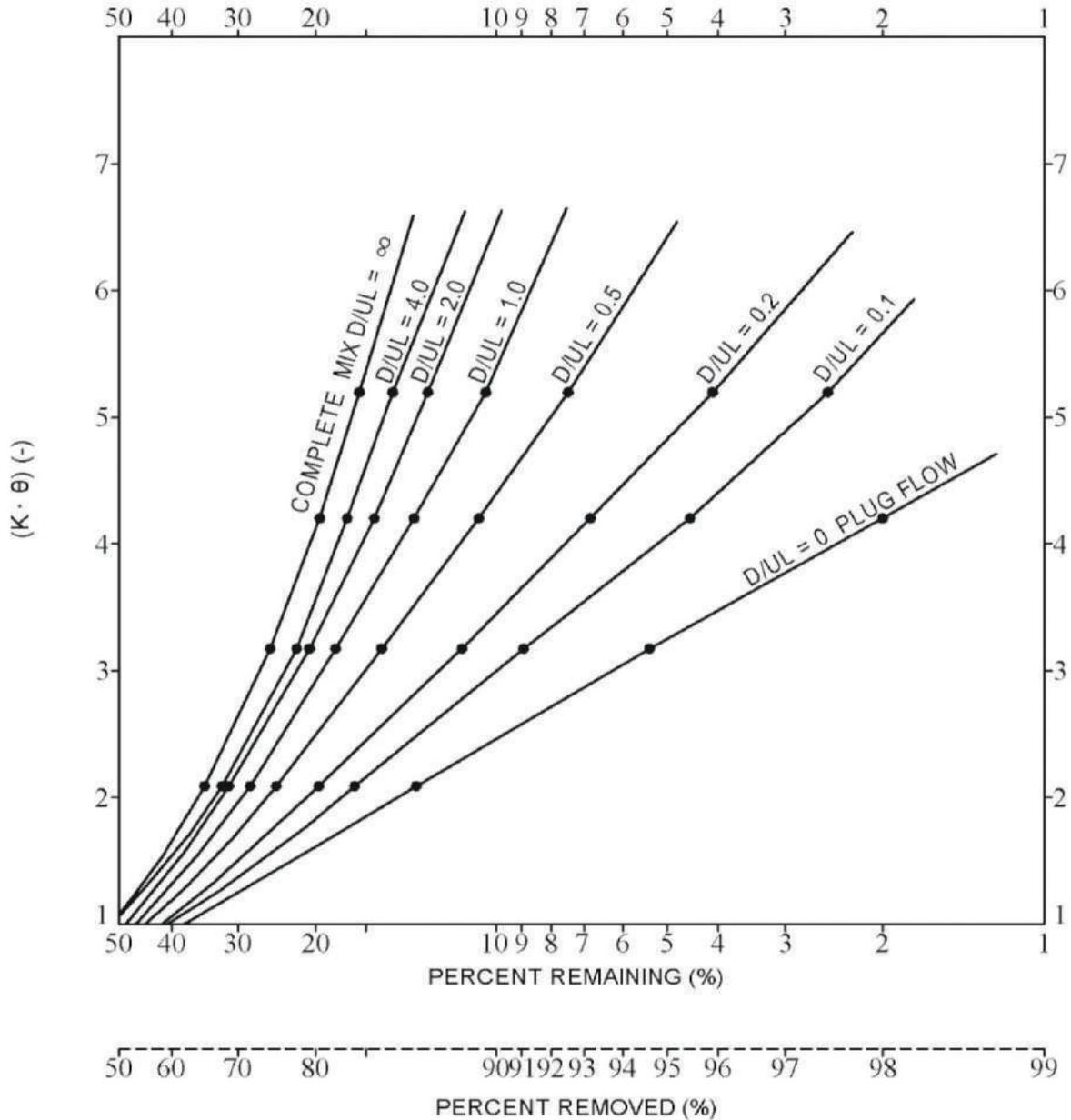
Source: CPHEEO, 1993

Figure 5.41 Substrate removal efficiency using the dispersed flow model (Wehner - Wilhem equation)

5.8.1.7.11   Mixing Conditions

The mixing conditions in a lagoon are reflected by the term 'd' which is known as the "Dispersion Number" and equals (D/UL) or (D/L^2). It is affected by various factors. Observed results have shown the (D/UL) values to be in the approximate range given in Table 5.13 (overleaf) for different length-width ratios of lagoons. By suitable choice of a lagoon's geometry one can promote either more plug flow or more complete mixing type of conditions. In case of cells in series, each cell may be well mixed with value of D/UL approaching 3.0 or 4.0 but overall the arrangements would give a relatively plug-flow type arrangement.

Table 5.13 Likely Values of Dispersion Numbers D/UL at Different Length –Width Ratios

Aerated Lagoon	Approximate range of D/UL values	Typical mixing condition
Length to width ratio 1:1 to 4:1	3.0 to 4.0 and over	Well mixed
Length to width ratio 8:1 or more	0.2 - 0.6	Approaching plug flow
Two or Three cells in series	0.2 – 0.6 (overall)	- do-

Source: CPHEEO, 1993

The values of D/UL can be determined by conducting dye (tracer) tests on existing units using well-known methods, but where D/UL values are required for design purposes prior to construction; they can be estimated either from lab-scale models or by using empirical equations. Low values of D/UL signify plug flow conditions and generally give higher efficiencies of substrate removal whereas the converse is the case with higher values of D/UL. However, process efficiency is not the only consideration; process stability under fluctuating inflow quality and quantity conditions, has also to be kept in view. For municipal or domestic sewage, relatively plug flow type conditions (i.e. low values of D/UL) are preferred. In case of industrial wastes, relatively well mixed condition may be preferred (i.e. higher values D/UL) depending upon the nature of the industrial waste; the greater the fluctuations in quality and quantity of industrial wastes, the greater the advantage in adopting well-mixed conditions. Figure 5.42 gives some examples of different types of arrangements using baffles or cells in series.
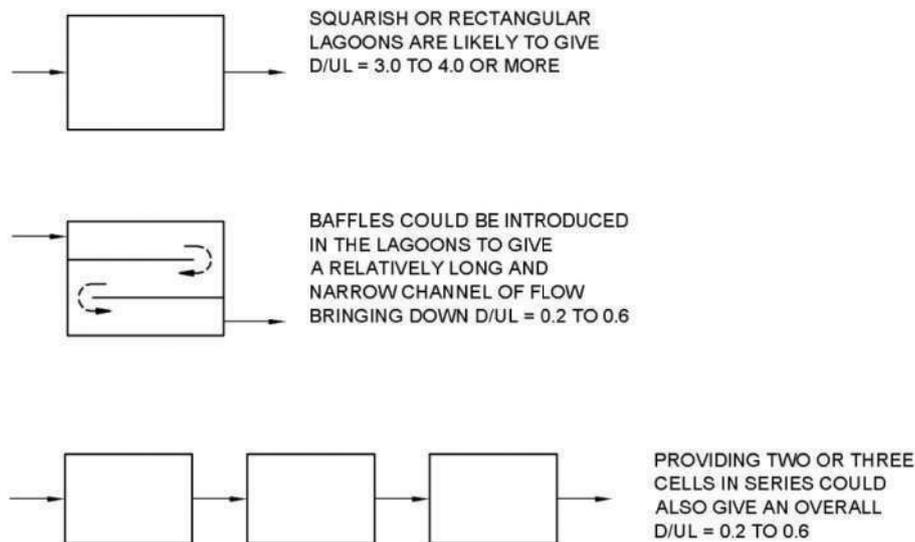
Source: CPHEEO, 1993

Figure 5.42 Estimated Effect of Lagoon Geometry D/UL

Lagoons are generally rectangular though it is not particularly essential. Natural land contours may be followed to the extent possible to save on earthwork. Lagoon units may be built with different length-width ratios and arrangement of internal baffles to promote desired mixing conditions. Lagoons may also be provided as two or three stage systems with the subsequent units placed at a lower level than the first if desired.

The construction techniques for aerated lagoons are similar to those used in case of oxidation ponds with earthen embankments. Pitching of the embankment is desirable to protect it against erosion. In cases where soil percolation is expected, suitable lining may have to be provided to maintain the design level in the lagoon and avoid ground water pollution.

5.8.1.7.12   Substrate Removal Rates

As shown in Table 5-12 for facultative aerated lagoons the overall substrate removal rate K for sewage varies from 0.6-0.8 per day (soluble BOD basis) at 20ºC. At other temperatures in lagoons the values are obtained from:

$$(K)_{T°C} = (K)_{20°C} \times 1.035^{(T-20)} \tag{5.33}$$

The temperature in a lagoon T_L is estimated from the following equation:

$$\frac{\theta}{h} = \frac{T_i - T_L}{f(T_L - T_a)} \tag{5.34}$$

Where

Θ = detention time in days,

h = depth of lagoon in meters,

T_i and T_a are the temperatures (°C) of influent sewage and ambient air respectively and

f = the heat transfer coefficient for aerated lagoons which is 0.49 m/day.

The average winter month temperature is critical for determining the detention time required. As stated earlier, the detention time to be provided in a lagoon can be determined from Equation (5.32) or Figure 5.42 for any desired efficiency for the computed temperature and mixing conditions in the lagoon.

5.8.1.7.13   Power Level

The power input in facultative aerated lagoons has to be adequate only to diffuse dissolved oxygen uniformly in the system and no effort is made to keep the solids in suspension. Hence, a minimum power level of 0.75 watts per cum lagoon volume should be adequate, but this should be checked with the aeration equipment supplier for its oxygenation characteristics and compatibility with proposed depth and shape of lagoon. For treating sewage the power requirement varies from 12 to 15 kWh/person/year or 2 to 2.5 HP per 1,000 population equivalent.

The oxygenation capacity of aerators is reported to range from 1.87 to 2.0 kg Oxygen/kWh at standard conditions for power delivered at shaft. Spacing of aerators should be adequate for uniform aeration all over the lagoon area without much overlap of the circle of influence of adjoining aerators as specified by the manufacturers. A minimum of two aerators would be desirable to provide the required make up for the total power requirement.

Aerators ranging from 3 HP to 75 HP are now readily available in the country. They can be either floating or fixed type. Floating aerators are mounted on pontoons which should be corrosion-free and which have the advantage of being able to adjust themselves to actual levels obtaining in the lagoons due to seepage and / or fluctuating inflows. Fixed aerators are mounted on columns and levelled with regard to the outlet weir level to ensure required submergence.

5.8.1.7.14   Effluent Characteristics

The effluent is generally made to flow over an outlet weir. As the concentration of solids passing out in the effluent may be nearly the same as that in the lagoon, the BOD corresponding to the volatile fraction of these solids (assumed as 0.77 mg per mg VSS in effluent) should be added to the value of the soluble BOD, obtained by use of Equation (5.32) or Figure 5-41. Thus, the final effluent BOD is given by:

Final BOD, mg/l = Soluble BOD, mg/l + (0.77) (VSS in effluent), mg/l (5.35)

It is because of the suspended solids (expected to range from 40 to 60 mg/l in case of sewage in the final effluent that the total effluent BOD is difficult to reduce below 30-40 mg/l in winter. At other times of the year BOD of less than 30 mg/l may be possible. This range of BOD is more than adequate for irrigation purposes. For river discharge, the applicable standards should be ascertained and design made accordingly, Where necessary, further reduction of BOD can be achieved either by a small increase in detention time or by more efficient interception of solids flowing out (e.g. deeper baffle plate ahead of outlet weir) or by provision of an additional treatment unit. Nitrification is not likely to occur in aerated lagoons, Coliform removal shows considerable seasonal variation (60 - 90% removal).

5.8.1.7.15   Sludge Accumulation

Sludge accumulation occurs at the rate of 0.03 to 0.05 cum per person per year as in the case of oxidation ponds and is manually removed once in 5-10 years and used as good agricultural soil filler. The depth of the lagoon may be increased a little to allow for sludge accumulation, if desired.

5.8.1.7.16   Conclusion

The removal efficiencies in terms of power input are comparable to some of the other aerobic treatment methods seen earlier in this chapter but the greatest advantage with aerated lagoons lies in their simplicity and ruggedness in operation, the only moving piece of equipment being the aerator. Civil construction is mainly earthwork, and land requirement is not excessive.

5.8.2   Attached Growth Systems

5.8.2.1   Historical Development of Attached Growth in Rock Media Filters

The earliest known attached growth systems were cases of raw sewage cascading over rock beds in river courses and microbes growing over the rock surfaces thus bringing about a variety of aerobic metabolism in the upper layers, anaerobic metabolism in the benthic layers and facultative metabolism in the intermediate sections.

The nearest to this can be seen in Rajneesh Ashram in Pune, where the raw sewage of a nearby economically weaker section habitation is diverted into a similar manmade cascading nullah and a light forestry is grown on both sides to encourage evapotranspiration. There are locations where benches have been put up along the course where people can sit and no foul odour perceptible. At the end of the nullah, the treated sewage is clear, odourless and colourless and is pumped over a mini rock built water fall. It is aesthetically a acceptable quality perhaps used downstream for agriculture and horticulture. Scientific data is not available on actual quality improvement, but is an accepted solution by the local population in the absence of an organized STP.

5.8.2.2   Development of Modern Synthetic Media

The trickling filter and intermittent sand filters are the earliest treatment processes. The trickling filter media was rock media of about 100 mm to 150 mm stones loosely placed by hand. Design data were evolved for these trickling filters by compiling the data on their performances especially in the USA and the famous National Research Council (NRC) and Ten State Standards were purely mathematical equations of best fit of the data. Eckenfelder Sr and Rankin were some authors who postulated theoretical approaches and these were used in some situations. However, with passage of time, the stone media has been given up the world over. In India also, the largest known installation at Piranha sewage farm has been since converted to ASP. The reasons were mainly the clogging and choking of the flow channels between the rocks and under drains due to the slow erosion of the stone and due to microbial corrosion and attrition. Consequently it was needed to physically remove and repack the whole filter volume of rocks. The rotary reverse jet distributor also created its own problems due to grit settling in the arm ducts and the turn-table immediately being stuck. This differential weight crushed the ball race and twisted the ball retainer rings in the turn-table. This has prompted innovations whereby light weight synthetic media of much higher surface area per unit volume have come up in the market.

The variations in physical arrangement are:

a) Fixed Film Reactors (FFR) which are attached growth on fixed film on stationary media and the applied sewage trickles down the exposed surfaces of the media.

b) Submerged Fixed Bed Reactors (SFBR) are attached growth on fixed film on submerged stationary media in the reactor and sewage flows through the media upward. The commercially known technologies as Submerged Aeration Fixed Film (SAFF), Rotating Biological Contactor (RBC), Fixed Bed Biofilm Activated Sludge Process (FBAS), etc., come under this and can again be either aerobic or anaerobic.

In both these cases, the microbes grow on the surfaces of the media and increase in thickness by the subsequent microbes adhering on the previous film and once the thickness becomes weighty the microbial film sloughs off the media and fresh microbes start developing. This is a cyclic process and the degree of organic matter removal can be intermittently fluctuating.

The advantage of the FFR is that huge liquid retaining reactors like aeration tanks need not be constructed and the FFR can be designed and constructed as simple silos and save the energy for mixing the contents.

These have to be structurally designed as liquid retaining structures because; it is necessary to flush the media periodically when organisms have grown too much and choke the flow pathways. The flushing is done by filling the entire height of the reactor with sewage after closing the outlet gate and suddenly draining it by opening the gate.

5.8.2.3   Physical Features

5.8.2.3.1   Shape of Reactors

Reactors may have circular, rectangular or square shape. Fixed nozzles or nozzles mounted on moveable arms are used for flow distribution. Rectangular square or circular shapes are used and the circular shape has the advantage of structural economy.

5.8.2.3.2   Provision for Flooding

This is needed in the case of FFR. Provision for intentional flooding and sudden draining of the reactors is useful for controlling filter flies and ponding. To enable flooding, the reactor walls must be designed for the internal water pressure and the main collecting channel must be placed inside the filter and provided with gate valves. An overflow pipe leading from the filter to the main collecting channel downstream from the gate valve is also necessary. Provision for filter flooding should always be made in the case of small reactors. Such a provision in large reactors would not only increase the cost but also cause hydraulic problems with the sudden discharge of large volumes of sewage when the flooded reactor is drained. In such cases alternate methods are needed. These can be pumped spray of treated sewage by hosing it and also air scouring simultaneously from the bottom.

5.8.2.3.3   Side Walls

The side walls of FFR and SFBR shall be RCC or brickwork subject to structural requirements of water pressure on side walls.

5.8.2.3.4   Floor

The floor is designed to support the under-drainage system and the superimposed filter media. The usual practice is to provide an RCC slab over a proper levelling course with slope between 0.5% and 5% towards the main collecting channel. The flatter slopes are used in larger reactors. The floor shall permit installation of fixed air headers for fixing diffuser elements therein and provision for gate controlled draining of the reactor.

5.8.2.3.5   Under Drainage System for FFR

5.8.2.3.5.1   Slope of Under Drains

The under drainage system is intended to collect the trickling sewage and sloughed solids and to convey them to the main collecting channel and to ventilate the media. The under drain covers the entire floor of the reactor to form a false bottom and consists of drains with semi-circular or equivalent inverts.

They can be formed of precast vitrified clay or concrete blocks, complete with perforated cover or they may be formed in-situ with concrete or brick and covered with perforated precast concrete slabs. The slope of the under drain should be the same as that of the floor. The drains shall be so sized that flow occupies less than 50% of the vertical cross-sectional area with velocities not less than 0.6 m/s at average design flow. The cover over the drains shall be perforated to provide a total area of not less than 15% of the surface area of the filter as inlet openings into the drains. The under drains may be open at both ends so that they may be inspected easily and flushed out if they become clogged.

5.8.2.3.5.2   Main Collecting Channel

The main collecting channel is provided to carry away the flow from the under drains and to admit air to the reactor. In a circular reactor, the main channel may be located along the diameter with a slight offset from the centre. Alternatively the channel may be provided along the outer periphery of the reactor. If inside the reactor, the channel shall be provided with perforated covers to enable drainage and ventilation of the reactor media above the channel. The channel should be extended outside the reactor, both at the upper end and lower ends with vented manholes to facilitate ventilation and access for cleaning. The channels shall have semi-circular or other rounded inverts. The velocity in the channels shall not be less than 0.6 m/s for the average hydraulic loading. The flow shall be only half-depth particularly where recirculation is low. At the peak instantaneous hydraulic loading, the water level in the channel should not rise above the inverts of the under drains at their junctions with the channel.

5.8.2.3.5.3   Ventilation

Adequate natural ventilation can be ensured by proper design of the under drains and effluent channels. For reactors larger than 30 m dia., a peripheral head channel on the inside of the reactor with vertical vents is desirable to improve ventilation. One m^2 of open grating in ventilating manholes and vent stacks should be provided for 250 m^2 of reactor area. The vertical vents can also be used for flushing the under drains. In extremely deep or heavily loaded reactors there may be some advantage in forced ventilation if it is properly designed, installed and operated. Such a design should provide for air flow of one m^3/min/m^2 of reactor area in either direction. It may be necessary during periods of extremely low air temperature to restrict the flow of air through the reactor to keep it from freezing. However a minimum air flow of 0.1 m^3/min/m^2 of reactor area should be provided.

5.8.2.3.5.4   Reactor Media

The requirements for reactor media are high specific surface area, high percentage of void space, resistance to abrasion and good structural strength to withstand deformation during placement, insolubility in sewage and resistance to spalling and flaking. The media shall be of virgin material of PVC or PE or HDPE. Recycled materials shall not be used.

5.8.2.3.5.5   Synthetic Media

Synthetic reactor media have of late been used successfully in super rate reactors for the treatment of strong industrial wastes or sewage mixed with strong industrial wastes.

The hydraulic loading rates are between 40 to 200 $m^3/d/m^2$ and organic loading rates between 0.8 to 6.0 kg BOD/d/m^3. The media consists of interlocking sheets of plastics which are arranged in a honeycomb fashion to produce a porous and non-clog reactor media. The sheets are corrugated so that a strong, lightweight media pack is obtained. Reactors as deep as 12 m are reported to have been used with this type of synthetic media.

5.8.2.3.5.6   Reactor Dosing

In the case of low rate reactors, the minimum flow rate of sewage inflow may not be sufficient to rotate the distributor and discharge sewage from all nozzles. Hence, when adequate head is available dosing tank is provided to collect the settled sewage and dose the reactor through a siphon intermittently. When head is not adequate, a collection well and pump is provided. The dosing siphons are designed to dose the reactors once in about 5 minutes under average flow conditions. In the case of high rate reactors, there is no need for the special dosing device since continuous dosing is possible.

5.8.2.3.5.7   Flow Distribution

Fixed nozzle distributors are not preferred because of the elaborate piping requirement and the necessity of dosing tanks, siphons or motor operated valves to obtain variable dosing rates. Moreover, physical access to each nozzle for cleaning requires the operator to walk over the slippery slime on the media top surfaces which is risky. Among the moving types, the longitudinally travelling distributors with limit switches at each end is a solution but the inlet arrangements from a fixed discharge location to the moving off take of the distributor arms is a challenge for design and upkeep needing bellows etc. The alternative practice is the reverse jet rotary distributors which generate the propulsion by the reverse jets on opposite diametrical arms, but here again, the problems cited already in Section 5.8.2.2 are important. The modern method is to facilitate a peripheral electrically operated drive similar to edge driven bridges of clarifiers and these are commercially available in the country up to 60 m dia. The central feed pipe to the well of the distributor is generally taken up from below the reactor floor or just above the under drains and through the reactor media. The pipe should be designed for a peak velocity of not greater than 2.0 m/s and an average velocity not less than 1 m/s.

The reaction type rotary distributor consists of a feed column at the centre of the reactor, a turn table assembly and two or more hollow radial distributor arms with orifices. The turntable should be provided with anti-tilt devices and arrangements for correcting the alignment to obtain balanced rotation. The turntable assembly is provided with a mercury or mechanical water seal at its base. The current trend is to discourage mercury seals because of the chances of causing mercury pollution. Facilities should be available for draining the central column of the flow distributor for attending to repairs and maintenance.

The distributor arms are generally two in number and multiples of two are being adopted. When multiple arms are provided, low flows are distributed through two arms only and as flow increases, it is distributed by the additional arms. This is achieved by overflows from weirs incorporated in the central column diverting the higher flows into the additional arms. The peak velocities in the distributor arms should not exceed 1.2 m/s.

The distributor arms are generally fabricated of steel and are liable to rapid corrosion. They should be fabricated and bolted together in such lengths as to facilitate dismantling for periodic repainting of their inside surfaces. The orifices in the distributor arms should be of light weight aluminium. Spreader plates, preferably of aluminium, should be provided below the orifices to spread out the discharge. The clearance between the distributor pipe and the top of the reactor media should be greater than 15 cm.

Distributor arms should have gates at the end for flushing them. At least one end plate should have arrangement for a jet impinging on the side wall to flush out fly larvae. The distributor arms may be of constant cross section for small units but in larger units and they are tapered from the centre towards the end to maintain the minimum velocity required in the arms.

The distribution arrangements should ensure uniform distribution of the sewage over the reactor surface for which the size and spacing of the orifices in the distributor arms have to be varied carefully from the centre towards the end. Under average flow conditions, the rate of dosing per unit area at any one point in a reactor should be within 10% of the calculated average dosing rate per unit area for the whole reactor. The distributors should also ensure that the entire surface of the reactor is wetted and no area is left dry.

Reaction type rotary distributors require adequate hydraulic head for operation. The head required is generally 1 to 1.5 m measured from the centre line of distribution arms to the low water level in the distribution well or the siphon dosing tank preceding the reactor. Alternatively, the rotary distributor driven by electric motor may be used and this type is particularly advantageous where adequate head is not available. The rotary speed shall ensure intervals of successive closings are between 15 and 20 seconds.

5.8.2.3.5.8   Pumping Arrangements

In a high rate reactor, pumping is required for recirculation. Pumping may also be required for lifting the reactor effluent to the clarifier or to the next stage reactor. Except in the case of small plants, recirculation pumps should be installed in multiple units so that the rate of recirculation rate can he changed as found necessary. Pumps for lifting the flow-through sewage should have adequate capacity to pump the peak flows through the plant. The pumps should be installed in multiple units to take care of diurnal variations which will approximately be the same as the sewage inflow to the plant. It will further be necessary to provide storage in the suction well equal to about 10 min of discharge capacity of the lowest duty pump. Float control arrangements are desirable in the suction well for controlling the number of pumps in operation. In all the cases, at least one pump should be provided extra as a standby. Furthermore, in the case of recirculation pumps, flow measuring and recording devices are desirable on the discharge line so that a record can be kept of the recirculation flow.

5.8.2.3.5.9   Ponding Problems

Ponding or clogging of the reactor media is one of the important operational problems in these submerged type reactors. Ponding decreases reactor ventilation, reduces the effective volume of the reactor and lowers the reactor efficiency. The ponding or clogging is due to excessive organic loading, inadequate hydraulic loading and inadequate size of media.

The remedies consist of raking or forking the reactor surface, washing the reactor by applying a high pressure stream of treated sewage at the surface, stopping the distributor to allow continuous heavy point by point dosing or chlorinating the influent with a dose not exceeding 5 kg/100 m^2 of reactor area.

Reactor flies pose another serious operational problem. The problem is more intense in the case of low rate reactors and in high rate reactors, fly breeding occurs mainly on the inside walls of the reactor. The problem can be reduced by (a) removing excessive biological growth by the previously discussed methods (b) flooding the reactor for 24 hours at weekly or bi-weekly intervals, (c) jetting down the inside walls of the reactor with a high pressure hose, (d) chlorinating the influent (0.5 to 1.0 mg/l) for several hours at one to two week intervals and (e) applying insecticides. The insecticide should be applied to the reactor side wall surfaces at intervals of 4-6 weeks. Development of resistant strains should be guarded against.

Reactor odour also presents a problem in these reactors. The odours are most serious when treating septic effluents in low rate reactors. They can be controlled by providing recirculation and maintaining a well-ventilated reactor.

In conditions of extreme cold weather, ice cover may form on the surface of the bed. Reduction of the recirculation flow, adjustment of nozzles or construction of wind breakers are methods used to reduce icing problems.

5.8.2.3.5.10   Multiple Units

In a single stage plant, it is advisable to split the required reactor volume into two or more units so that when one reactor is taken out of operation for maintenance or repairs, the entire sewage can be passed through the remaining units, overloading them temporarily. In a two stage plant, if multiple units are proposed in each stage, the entire sewage may be routed through the remaining units of the stage when one reactor in that stage is taken out of operation. However, the recirculation flow is maintained at the original level and operating the stage at a lower recirculation ratio. If, instead, only one reactor is proposed for each stage a bypass should be provided for each stage. It is customary in the design of two stage reactors to use two reactors of equal size.

5.8.2.3.5.11   Plant Hydraulics

The feed pipe to the reactor, the distributor, the under drains and the main collection channel should be designed for the peak instantaneous hydraulic loading on the reactor. In low rate reactors, the peak loading will be the peak discharging capacity of the dosing siphon or the dosing pump. In the case of high rate reactors, the peak loading on the reactors will be the sum of the peak rate of sewage flow and the constant recirculation rate. When multiple units are used for the high rate reactors in any stage, the hydraulics of the plant should be checked for peak loading with one reactor out of operation and the entire flow routed through the remaining units. A reduced recirculation ratio is adopted for this condition to reduce the peak loading and avoid over sizing of the piping.

When multiple units are used care should be taken to ensure that the flow is divided properly between the reactors.

5.8.2.4   Design Guidelines

Arising from the position that each media manufacturer claim their own values of surface area to unit volume of their media, the entire design guidelines being specified as a generic guideline for all these attached growth units becomes a challenge. At the same time, it is necessary not to lose track of the advantages of this technology. The real problem arises in specifying the estimate cost of these while initially preparing the same for according the procedural requirements of DPR and technical sanction for drafting the technical specifications in the tender document.

Until a relative evaluation of the already functioning reactors is completed and a generic set of design guidelines are formulated on a country basis, the design is best left to the tenderer in so far as the unit sizing and associated civil mechanical and piping details are concerned.

Tenders can still be invited and decided based on Section 5.19 titled addressing the recent technologies in choice of STPs.

5.8.3   Treatment Methods using Immobilization Carrier

5.8.3.1   Historical

As an improvement over the attached growth systems, the concept of trapping the microbes into the attached media without allowing them to slough off and keeping the media itself in fluidized state and thus improving on the consistency of the organic removal has been developed. This has been brought under the generic name of immobilized carriers. As otherwise, this technology is also schematically referred to as Moving bed biofilm reactors (MBBR) or Fluidized aerobic bed (FAB). The main reason for this technology to be attractive is its ability to reduce the waste sludge volumes.

5.8.3.2   Specific Microbiology

It has been shown that maintaining high biomass concentration and long solids retention time in a biological reactor can limit the waste sludge production for a given reduction of BOD. This is due to the higher biomass concentration in the reactor due to the immobilized biomass and hence the Food/Microorganism ratio going beyond the extended aeration. It is stated that during aeration, the synthesis and accumulation of readily biodegradable storage compounds are observed and these can be used for denitrification under starvation conditions.

5.8.3.3   Status

Enhancing active biomass concentration, prolonging the life of immobilized carrier and improving the stability of immobilized microorganism play important roles in the process efficiency. The construction, operation, preventing clogging and reducing renewal costs are challenges in the commercial engineering of this technology. However the fact remains that there are commercially operating STPs built with this technology in our country using various patented media of the respective vendors and with their own design criteria. As such, this technology holds the potential of reducing the footprint of the STP especially in land locked high density urban centres and thus merits its relative consideration.

5.8.3.4   Physical Features

5.8.3.4.1   Shape of Reactors

Reactors are usually like aeration tanks and the circular shapes find better acceptance due to reduced civil construction costs and also permitting tall structures as long as air cooled compressors can be installed to pump the air against such high heads of water column.

5.8.3.4.2   Floor

The floor shall permit installation of fixed air headers for fixing diffuser elements thereon inside the reactor and provision for gate controlled draining of the reactor.

5.8.3.4.3   Reactor Media

The requirements for reactor media are high specific surface area, high percent void space, resistance to abrasion or disintegration during placement, insolubility in sewage and resistance to spalling and flaking. The inbuilt configuration must permit hydraulic self-cleaning of the media itself and thereby safeguarding the need to take the reactor out of service to attend to cleaning the clogged media.

5.8.3.4.4   Netting to Hold Back Media

This is an important requirement and is usually provided near the top outlet of the treated sewage in the form of spread out netting across the entire plan area or a netted cowl around the off take of the inlet pipe. Care is needed periodically to renew these.

5.8.3.4.5   Auxiliary Mixers in the Reactor

The specific gravity of the media shall have to permit its floatation but at the same time the required value for keeping it submerged. It will be advantageous to install side wall mounted slow moving propeller blade mixers which can plummet the media downwards and thereby they can rise back and again be plummeted downward and thus ensure optimum contact between all media and the sewage.

This will prevent chances of microbes building up on the floor due to lack of transport velocities.

5.8.3.4.6   Multiple Units

The same guidelines as in section 5.8.2 on attached growth systems apply here also.

5.8.3.4.7   Plant Hydraulics

The same guidelines as in section 5.8.2 on attached growth systems apply here also.

5.8.3.5   Design Guidelines

The same guidelines as in section 5.8.2 on attached growth systems apply here also.

5.8.4   Stabilization Ponds

5.8.4.1   General

Stabilization ponds are open, flow-through earthen basins designed and constructed to treat sewage and provide comparatively long detention periods extending from a few to several days. During this period the organic matter in sewage is stabilized in the pond through a symbiotic relationship as illustrated in Figure 5.3 earlier. Lightly loaded ponds are also used as a tertiary step in sewage treatment for polishing of secondary effluents and destruction of coliform organisms and are called maturation ponds. In warm climate countries, the pond systems are cheaper to construct and operate compared to conventional methods. They also do not require skilled operational staff and their performance does not fluctuate from day to day. The only disadvantage of pond systems is the relatively large land that they require, but this is sometimes over-emphasized. In addition, land on the outskirts of a growing city can be a worthwhile investment. Pond systems must be considered as an alternative when treatment of sewage or upgrading of existing facilities are planned and the life time costs of various other treatment system should be calculated and compared.

5.8.4.2   Classification

5.8.4.2.1   Aerobic

Aerobic ponds are designed to maintain completely aerobic conditions. The ponds are kept shallow with depth less than 0.5 m and BOD loadings are 40 to120 kg/ha.d. The pond contents may be periodically mixed by float mounted paddle mixers. Ponds like these give rise to intense algal growth and have been used only on experimental basis

5.8.4.2.2   Anaerobic

Completely anaerobic ponds are used as pretreatment sometimes for municipal sewage. They are also used for digestion of STP sludge. Depending on temperature and waste characteristics, BOD load of 400 to 3000 kg/ha.d and 5 to 50 day detention period would result in 50 to 85% BOD reduction. Such ponds are constructed with a depth of 2.5 to 5 m to conserve heat and reduce land area. They have an odour problem due to sulphide gases.

5.8.4.2.3   Facultative

The facultative pond functions aerobically at the surface while anaerobic conditions prevail at the bottom. The aerobic upper layer oxidizes the sulphide gases and avoid the foul odours. The treatment effected is comparable to that of conventional secondary treatment processes. The facultative pond is suited and commonly used and further discussion in this chapter is therefore, confined to facultative ponds.

5.8.4.3   Mechanism of Purification

The physical, chemical and biological reactions in engineered pond systems are controlled by the design criteria. The functioning of a facultative stabilization pond and symbiotic relationship in the pond are shown schematically in Figure 5-3.

Sewage organics are stabilized by both aerobic and anaerobic reactions. In the top aerobic layer, where oxygen is supplied through algal photosynthesis, the non-settleable and dissolved organic matter in the incoming sewage is oxidized to carbon dioxide and water. In addition, some of the end products of partial anaerobic decomposition such as volatile acids and alcohols, which may permeate to upper layers, are also oxidized aerobically. The settled sludge mass originating from raw waste and microbial synthesis in the aerobic layer and dissolved and suspended organics in the bottom layers undergo stabilization through conversion to methane which escapes the pond in form of bubbles. For each kg of BOD-ultimate stabilized in this manner, 0.25 kg or 0.35 m^3 of methane is formed. Another reaction which sometimes occurs in the anaerobic layers is conversion of hydrogen sulphide to sulphur by photo-synthetic bacteria; if present in sufficient numbers they give a distinct pink hue to the pond appearance.

5.8.4.3.1   Aerobic and Anaerobic Reactions

The depth of aerobic layer in a facultative pond is a function of solar radiation, waste characteristics, loading and temperature. As the organic loading is increased, oxygen production by algae falls short of the oxygen requirement and the depth of aerobic layer decreases. Oxygen diffusing from top layers is utilized quickly and completely. Further, there is a decrease in the photo-synthetic activity of algae because of greater turbidity and inhibitory effect of higher concentration of organic matter.

Gasification of organic matter to methane is carried out in distinct steps of acid production by acid forming bacteria and acid utilization by methane bacteria. Production of methane is fundamental to BOD reduction by anaerobic metabolism. If the second step does not proceed satisfactorily there is an accumulation of organic acids in the pond bottom which diffuses towards the top layers. Furthermore, under such conditions the pH of the bottom layers may go down. This would result in complete inhibition of methane bacteria and the pond may turn completely anaerobic due to accumulation of end products of partial anaerobic decomposition, Imbalance between the activities of the two sets of microorganisms in a pond may result from two possible reasons. The waste may contain inhibitory substances which would retard the activity of methane producing organisms and not affect the activity of acid producers to the same extent. In treatment of sewage such a condition, however, does not arise. The other reason for the imbalance may be a fall in the temperature of the pond. The activity of methane bacteria decreases much more rapidly with decreasing temperature as compared to the acid formers and gas production stops at temperatures lower than 15ºC. Thus, year round warm temperatures and sunshine provide an ideal environment for operation of the facultative stabilization ponds.

5.8.4.3.2   Diurnal Variations

Both the dissolved oxygen and pH of the pond are subject to diurnal variation due to photosynthetic activity of algae which is related to incident solar radiation. A high dissolved oxygen concentration up to about 4 times the saturation value may be observed in the afternoon hours. Simultaneously, the pH value may reach a maximum of 9.0 or more due to the conversion of carbon dioxide to oxygen. Towards the evening or in the night, when photosynthetic activity decreases or ceases, there is a gradual decrease in both dissolved oxygen and pH.

In properly designed ponds, the dissolved oxygen does not completely disappear from the top layers at any time. The increase of pH is beneficial as it increases the die off rate of faecal bacteria like coliforms.

5.8.4.3.3   Odour Control

In a facultative pond, the nuisance associated with anaerobic reactions is eliminated due to the presence of oxygen in the top layers. The foul smelling end products of anaerobic degradation which permeate to the top layers are oxidized in an aerobic environment. Furthermore, due to a high pH in top layers, compounds such as organic acids and hydrogen sulphide, which would otherwise volatilize from the surface of the pond and cause odour problems are ionized and held back in solution.

5.8.4.3.4   Algae

In stabilization ponds, the significant algae are green which include Chlorella, Scenedesumus, Hydrodictyon Chlamydomonas and Ankistrodesmus and blue-green algae which include Oscillatoria, Spirulina, Merismopedia and Anacystis. The Chlorella, Scendesmus and Hydrodictyon possess relatively high oxygen donation capacity per unit weight. However, it is not practical to promote the growth of any particular type of algae in a pond which will depend on such factors as temperature, characteristics of the waste and intensity of sunlight. Concentration of algae in a stabilization pond is usually in the range of 100 to 200 mg/l which gives the pond effluent a typical green colour. Floating blue-green algae mats may develop in ponds during summer months. They are undesirable since they restrict penetration of sunlight leading to reduction in depth of aerobic layer. They also encourage insect breeding.

5.8.4.4   Design Considerations

The facultative pond system, though simple to operate, is a complex ecosystem. It is only by experience and understanding of the reactions that rational criteria are evolved. Appendix A.5.14 presents an illustrative design.

5.8.4.4.1   Areal Organic Loading

The permissible areal organic loading for the pond expressed as kg BOD5/ha.d will depend on the minimum incidence of sunlight that can be expected at a location and on the percentage of the influent BOD that would have to be satisfied aerobically. Many different methods have been developed for determining the permissible area loading and two methods are discussed here, being

(a) The BIS has related the permissible loading to the latitude of the pond location to aerobically stabilize the organic matter and keep the pond odour free (Refer IS: 5611) and

(b) another based on field experience. The recommended loading rates are in Table 5.14 overleaf.

The values are applicable to towns at sea levels and locales where the sky is clear for nearly 75% of the days in a year.

Table 5.14 Permissible Organic Loadings at Different Latitudes

Latitude (N) degree	Organic loading Kg BOD/ha.d
36	150
32	175
28	200
24	225
20	250
16	275
12	300
8	325

Source: CPHEEO, 1993

The values of organic loading given in Table 5.14 may be modified for elevations above sea level by dividing by a factor of (1+0.003 EL) where EL is the elevation of the pond site above MSL in hundred meters. An increase in the pond area has to be made when the sky is clear for less than 75% of the days. For every 10% decrease in the sky clearance factor below 75%, the pond area may be increased by 3%.

Another design approach, based on field experience in warm climates relates the permissible area BOD loading to the ambient temperature on the assumption that temperature would depend on solar radiation:

$$L_0 = 20T - 120$$ (5.36)

where

   L_o = design organic load in kg BOD_5/ha/d and
   T = average temperature during coldest month of the year in degree Celsius.

The designs based on the two methods given above, as well as other methods developed empirically, wherever possible should be checked against field experience in the region. When the ponds are intended to serve small communities or when they are located close to residences, it will be prudent to adopt lesser BOD loading to fully ensure absence of odours.

5.8.4.4.2   Detention Time and Hydraulic Flow Regimes

The flow of sewage through a pond can approximate either plug flow or complete mixing, which are two extreme or ideal conditions. If BOD exertion is described by a first order reaction, the pond efficiency is given by the equation in the next page:

For Plug flow

$$\frac{L_e}{L_i} = e^{-K_l t}$$ (5.37)

For Complete Mixing

$$\frac{L_e}{L_i} = \frac{1}{1 + K_l t}$$ (5.38)

where

L_i and L_e = influent and effluent BOD respectively,
t = detention time,
K_l = BOD reaction rate constant.

The value of K_l varies between 0.05 and 0.2 per day and is independent of temperatures above 15ºC. The lower values were determined for secondary and tertiary ponds.

In practice the hydraulics lies between the two regimes and is described as dispersed flow. The efficiency of treatment for different degrees of intermixing, characterized by dispersion numbers, can be determined as given in Section 5.8.1.7.12 for aerated lagoons. Dispersion numbers are determined by tracer studies. Dispersion numbers for stabilization ponds vary from 0.3 to 1.0. Choice of a larger value for dispersion number or assumption of complete mixing would give a conservative design and is recommended.

5.8.4.4.3   Depth

Shallow depths in facultative ponds will allow the growth of aquatic weeds in the ponds. The optimum range of depth for facultative ponds is 1.0 - 1.5 m. When depth determined from area and detention period works out lesser than 1.0 m, the depth should be increased to 1.0 m, keeping surface area unchanged.

5.8.4.4.4   Sludge Accumulation

The rate of sludge accumulation in facultative ponds depends primarily on the suspended solids concentration in the sewage. It varies from 0.05 to 0.10 m^3/capita/year. A value of 0.07 m^3/capita/year forms a reasonable assumption in design. In multiple cell ponds operated in series, most of the sludge accumulation will be in the primary cells. Continued sludge accumulation in ponds over many years will cause (i) sludge carryover into the effluent, (ii) development of aquatic weeds, and (iii) reduction in pond efficiency due to reduction in the detention period, Facultative ponds therefore require periodical desludging at intervals ranging from 6 to 12 years.

5.8.4.4.5   Bacterial Reduction

Bacterial reduction in ponds is similar to BOD reduction except the BOD reduction rate constant is replaced by bacterial die off constant, Kb and inputs and outputs are in terms of bacterial concentrations Ni and Ne, respectively.

It is customary to use completely mixed conditions when calculating bacterial reduction. This gives a conservative design. Overall bacterial reduction in 'n' ponds of equal detention time 't' in series is given by

$$\frac{N_e}{N_i} = \frac{1}{(1+K_b t)^n}$$

(5.39)

A commonly used value of K_b for faecal bacteria at 20°C is 2.0 per day. The value of K_b at other temperatures may be calculated by

$$K_{b(T)} = K_{b(20)}(1.19)^{(T-20)}$$

(5.40)

where,

$K_{b(T)}$ and $K_{b(20)}$ are values of the constant at T and at 20°C respectively.

5.8.4.4.6   Mosquito Aspects

It is popularly believed that ponds will promote the growth of mosquitoes. This is not true. The stages of growth of mosquitoes are shown in Figure 5.43.
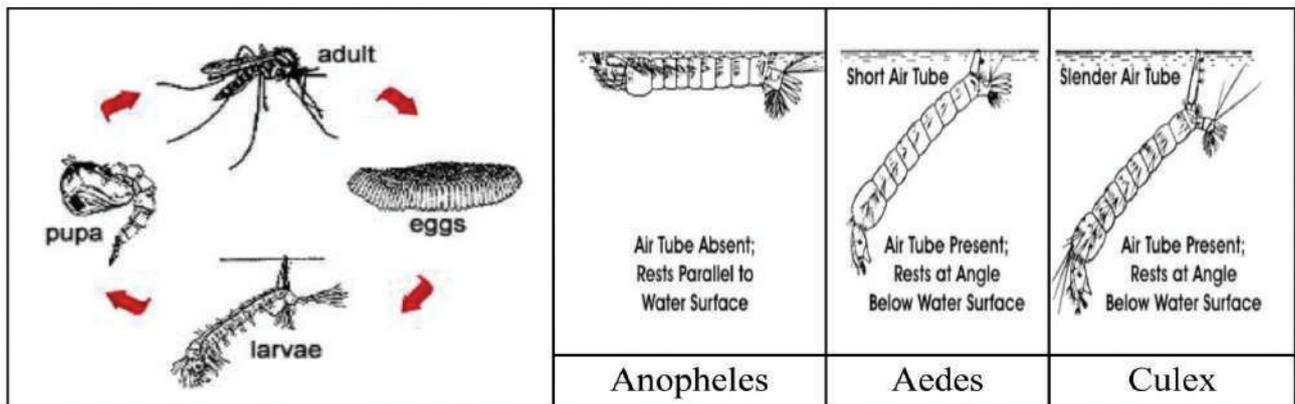
Figure 5.43 Stages of Mosquito growth and Larvae Stage Breathing near Water Surface

It may be seen that the mosquito eggs becomes larvae and they must breathe air to survive. This is why the larvae generally remain just beneath the surface of the water. There are three broad types of mosquito larvae like this. These are Anopheles, Aedes and Culex. The Anopheles lay parallel to the water's surface in order to get a supply of oxygen through a breathing opening. The other two types have siphon tubes and lay beneath the water surface and the siphon pipe punctures the water and protrudes into the air. After some days in the larval stage they grow to pupa and then adults. In a well operated pond free of any weeds, the wind causes the water surface to be continuously oscillating non-stop. Due to this, raw sewage gets into the breathing system of the larvae and chokes them. Thus the ponds do not promote mosquitoes. Hence, the pond shall be free of weeds which will otherwise prevent the oscillation of the water surface.

5.8.4.5   Construction Details

5.8.4.5.1   Site Selection

Facultative pond sites should be located as far away as practicable (at least 200 m) from habitations or from any area likely to be built up within a reasonable future period. If practicable the pond should be located such that the direction of prevailing wind is towards uninhabited areas. The pond location should be downhill of ground water supply source to avoid their chemical or bacterial pollution. Special attention is required in this regard and in porous soils and in fissured rock formations. The pond site should not be liable to flooding and the elevation of the site should permit the pond to discharge the effluent by gravity to the receiving streams. The site should preferably allow an unobstructed sweep of wind across the pond and open to the sun. Trees should not be grown in the bunds and for an annular distance of 10 m from the toe of the bunds. Advantages should be taken of natural depressions while locating the ponds.

5.8.4.5.2   Pre-treatment

Medium screens and grit removal devices shall be provided before facultative ponds.

5.8.4.5.3   Construction in Stages

In cases where the design flow will occur only after a long time, it is important to design facultative ponds in multiple cells and construct the cells in stages. Otherwise, the small flows in the initial years may not be able to maintain satisfactory water levels in the ponds. This will cause objectionable weed growths. The weeds will prevent the water surface from oscillation by wind. Hence mosquitoes can breed and multiply. Construction in stages will also reduce initial costs and help in planning future stages based on the performance data of the first stage.

5.8.4.5.4   Multiple Units

Multiple cells are recommended for all except small installations (0.5 ha or less). Multiple cells in parallel facilitate maintenance as any one unit can be taken out of operation temporarily for desludging or repairs without upsetting the entire treatment process. The parallel system also provides better distribution of settled solids. Multiple cells in series decrease dispersion number and enable better BOD and coliform removal and reduced algal concentration in the effluent. The series system implies a high BOD loading in the primary cells and to avoid anaerobic conditions in these cells, they should have 65% to 70% of the total surface area requirements. A parallel series system possesses the advantages of both parallel and series operations. A convenient arrangement for this system consists of three cells of equal area, of which two are in parallel and serve as primary ponds and the third serves as secondary pond in series. Individual cell should not exceed 20 ha in area.

5.8.4.5.5   Pond Shape

The shape should be such that there are no narrow or elongated portions. Rectangular ponds with length not exceeding three times the width are to be preferred. Maximum basin length of 750 m is generally adopted. The comers should always be rounded to minimize accumulations of floating matter and to avoid dead pockets.

5.8.4.5.6   Embankment

Ponds are usually constructed partly in excavation and partly in embankment. The volume of cutting and the volume of embankment should be balanced to the maximum extent possible in order to economize construction costs. Embankment materials usually consist of material excavated from the pond site. The material should be fairly impervious and free of vegetation and debris. The embankment should be compacted sufficiently. The top, width of the embankment should be at least 1.5 m to facilitate inspection and maintenance. The free board should be at least 0.5 m in ponds less than 0.5 ha in area. In larger installations, the free board should be designed for the probable wave heights and should be at least 1.0 m. Embankment slopes should be designed based on the nature of soil, height of embankment and protection proposed against erosion. Outer slopes are generally 2.0 to 2.5 horizontal to 1 vertical. Inner slopes are made 1.0 to 1.5 when the face is fully pitched and flatter and 2.0 to 3.0, when the face is unprotected. Inner slopes should not exceed 4 as flatter slopes create shallow areas conducive to the growth of aquatic weeds. The outer faces of the embankments should be protected against erosion by turfing. The inner faces should preferably be completely pitched to eliminate problems of erosion and growth of marginal vegetation. Pitching may be by rough stone revetment or with plain concrete slabs or flat stones with adequate gravel backing. When complete pitching is not possible, at least partial pitching from a height 0.3 m above water line to 0.3 m below water line is necessary and the face above the line of pitching should be turfed to the top of embankment. A properly constructed pond is shown in Figure 8.10 in this manual.

5.8.4.5.7   Pond Bottom

The pond bottom should be level, with finished elevations not more than 0.10 m from the average elevation. The bottom should be cleared of all vegetation and debris. The soil formation of the bottom should be relatively impervious to avoid excessive liquid losses due to seepage. Where the soil is loose, it should be well compacted. Gravel and fractured rock areas must be avoided.

5.8.4.5.8   Pond Inlets

The pipeline conveying raw sewage to the pond, whether by gravity or by pumping, should be terminated in a flow measuring chamber located close to the pond. There should be sufficient fall from the measuring chamber to the pond surface so that the measuring weir may not be submerged. If the pond installation is in multiple parallel cells, the measuring chamber should have flow splitting provision and there should be separate pipeline to each cell. The size of the pipeline may be designed to maintain an average velocity of 0.3 m/s. The pipeline should be semi-flexible and should be properly supported inside the pond. In case the pond cell is large, multiple inlets should be provided along the inlet side of the pond at the rate of one for every 0.5 to 1.0 hectare of pond area. This requirement applies also to outlets. In case the pond is small, a single inlet and a single outlet will be sufficient. The inlets in the pond shall be so located as to avoid short-circuiting of flow to the outlets. The inlets should not be upwind of the outlets and should be extended into the pond for one-third to one-fourth the pond length or 15 to 20 m, whichever is less. The discharge may be horizontal and at half depth. A concrete apron of adequate size should be provided under the discharge to prevent erosion of pond bottom, especially when the pond is being filled up.

5.8.4.5.9   Pond Outlets

Multiple outlets are desirable except in small ponds and may be provided at the same rate as for inlets, one for every 0.5 ha pond area. The outlets should be so located with reference to the inlets as to avoid short-circuiting. The outlet, structures may consist either of pipes projecting into the ponds or weir boxes. In the former case vertical tees and in the latter case hanging baffles submerged to a depth of 0.25 m below the wafer surface should be provided to ensure that floating algal scum is not drawn along with the effluent. When the outlet structure is a weir box, it is desirable to provide adjustable weir plates so that the operating depth in the pond can be altered if required. Where the pond effluent is to be used for farming and involves pumping, the outlet pipe should be led to a sump of adequate capacity (30 minutes at the rate of pumping). All piping should be provided with suitable valves to facilitate operation and maintenance.

5.8.4.5.10   Pond Interconnections

Pond interconnections are required when ponds are designed in multiple cells in series. These interconnections should be such that the effluent from one cell withdrawn from the aerobic zone can he introduced at the bottom of the next cell. Simple interconnections may be formed by pipes laid through the separating embankments. At their upstream ends, the interconnecting pipes should be submerged about 0.25 m below the water level. The downstream ends may be provided with a bend, facing downward, to avoid short-circuiting by thermal stratification, care being taken to prevent erosion of the embankment.

5.8.4.5.11   Other Aspects

Provision should be made for flow measurement both at inlet and outlet of the ponds, wherever practicable, facilities should be available to drain out the pond completely by gravity through a sluice arrangement. The pond site should be fenced to prevent entry of cattle and discourage trespassing. Public warning boards should also be put up near the ponds clearly indicating that the pond is a sewage treatment facility.

5.8.4.6   Performance

The algae in the pond effluent will exert BOD in the standard laboratory BOD test involving darkroom incubation and will give high SS values. The BOD and SS values may each be in the range of 50 to 100 mg/l. However, the effluent will not cause nuisance when disposed of on land or discharged into receiving waters because the algal cells do not readily decompose or exert oxygen demand under natural conditions, in fact, the algae increases the oxygen levels in the receiving water by continued photosynthesis.

Because of the above reasons, the standard BOD and SS tests are not considered useful for evaluating the quality of facultative pond effluents.

The quality is usually assessed based on the BOD_5 of the filtered effluent, the assumption being that the suspended solids in the effluent are all algae. The filtration procedure adopted for the test is the same as for the suspended solids test.

Well designed facultative ponds give about 80% to 90% BOD reduction based on the filtered BOD_5 of the effluent. Facultative ponds also effect high bacterial reduction, the efficiency being particularly high in multi cell ponds operated in series. Coliform and faecal streptococci removals are as high as 99.99%. Intestinal pathogens belonging to Salmonella and Shigella groups are reportedly eliminated in stabilization ponds. Cysts of Entamoeba Histolytica and Helminthic larvae are also eliminated.

5.8.4.7   Construction for Filtering Out Algae

The algae flowing out of the pond need not be removed when the treated sewage is used for crop irrigation. The most appropriate technique for this is a rock filter, which consists of a submerged porous rock bed within which algae settle out as the effluent flows through. The algae decompose releasing nutrients which are utilized by bacteria growing on the surface of the rocks. In addition to algal removal, significant ammonia removal may also take place through the activity of nitrifying bacteria growing on the surface of the filter medium. The performance depends on loading rate, temperature and rock size and shape. The permissible loading increases with temperature and in general an application rate of 1.0 m^3 of pond effluent per m^3 rock bed per day should be used. Rock size is important, as surface area for microbial film formation increases with decreasing rock size but, if the rocks are too small, then problems can occur with clogging. Rock size is normally 75 to 100 mm, with a bed depth of 1.5 to 2.0 m. A typical rock filter is shown in Figure 5.44.

Source. Duncan Mara

Figure 5.44 Rock Filter Installed in the Corner of a Pond at Veneta, Oregon, USA

The effluent should be introduced just below the surface layer because odour problems are sometimes encountered with cyanobacterial films developing on wet surface rocks exposed to the light. Construction costs are low and very little maintenance is required, although periodic cleaning to remove accumulated humus is necessary, but this can be carried out during the cooler months when algal concentrations are lowest.

5.8.4.8   Applications

The facultative pond is simple and economical to construct. It does not require skilled operation and is easy to maintain. Properly designed, the pond also gives consistently good performance. The facultative pond has therefore become very popular for sewage treatment. The method is suited wherever land is cheap and readily available and may be used for treating sewage either for discharge into streams or lakes or for use on land. The method is particularly useful for interim sewage treatment when due to lack of funds or due to meagre flow in the initial stages, it is considered inexpedient to construct initially the treatment plant envisaged ultimately. Their performance in terms of pathogen removal and reliability is high. The treated sewage can be used for agriculture in conformity with the quality stated in Table 7.19. In regard to pisciculture pl refer to section 4.15 of Part B manual.

5.8.5   Anaerobic Treatment of Sewage

5.8.5.1   Introduction

Anaerobic treatment of sewage has a number of advantages over aerobic treatment processes. These are (a) lesser electrical energy input of the system because oxygen is not required and hence, aeration is not needed and (b) Methane gas, which has a thermal value is produced from which electrical energy can be generated, However, there is a disadvantage in that the corrosive Hydrogen Sulphide gas is produced from the sulphate present in the sewage. Anaerobic digestion as a unit process in municipal sewage treatment has been in use since the beginning of this century. It is employed for stabilization of sludge solids from primary and secondary sedimentation tanks either in closed digesters or open lagoons. Anaerobic lagoons are also used for treatment of industrial wastes. Conventionally, the anaerobic process is considered a slow process requiring digesters of large hydraulic retention time (HRT). In recent years a number of high rate systems have been constructed to treat concentrated liquid industrial wastes and for direct treatment of municipal sewage. Application of anaerobic treatment technology, for treatment of municipal sewage has special significance in India because of high-energy savings and low capital and O&M and renewal and replacement costs. This section briefly reviews various high rate systems and summarizes the available design criteria. It also lists aspects of anaerobic treatment, which must be evaluated in the designs.

5.8.5.2   High Rate Anaerobic Systems

High rates of conversion of organics into methane can be obtained by maintaining a high concentration of microorganisms in a reactor and preventing them from escaping with the effluent. This concept is expressed as Sludge Retention Time (SRT) and is the ratio of mass of biological solids in the system to that escaping from the reactor. Maximal SRT is desirable for process stability, minimal sludge production and minimal reactor volume and thus reduces capital costs. Other requirements of high rate systems are intimate contact between incoming waste and the biological solids and maintenance of sufficiently warm temperatures.

Figure 5.45 (overleaf) shows the basic configurations of high rate anaerobic systems.
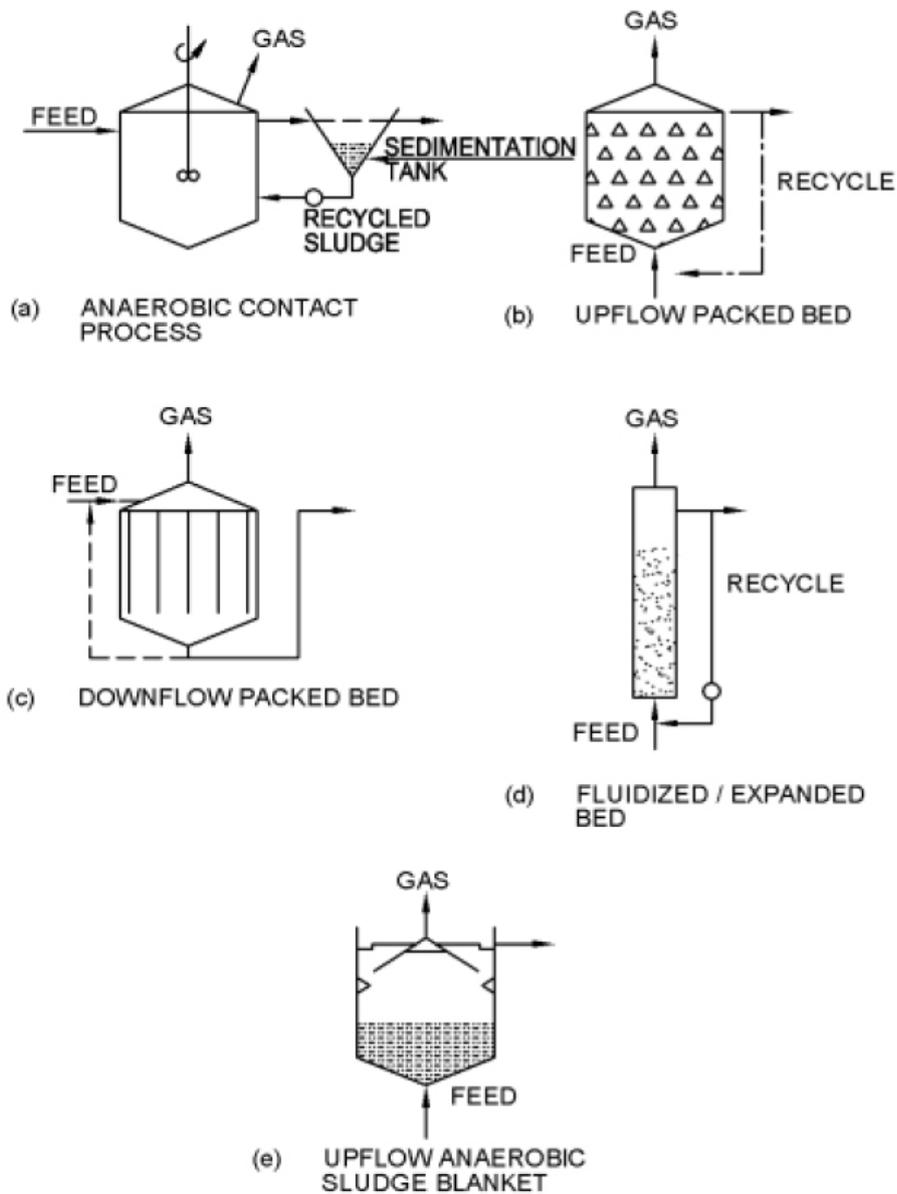
Source: CPHEEO, 1993

Figure 5.45 Configurations of High Rate Anaerobic Systems

5.8.5.2.1 Anaerobic Contact Process

The Anaerobic Contact process as in Figure 5.45 (a) is a stirred tank reactor in which the biomass leaving with the reactor effluent is settled in a sedimentation tank and recycled, thus increasing the SRT. The settling of the anaerobic sludge may at times be a limiting factor. Biomass separation may be improved using parallel plate separators. The process lends itself to concentrated wastes containing refractory suspended matter. Continuous and complete mixing in the reactor is not recommended, since this may adversely affect settling characteristics of the sludge, On the other hand, inadequate mixing may result in formation of dead zones inside the reactor. This process has been used for treatment of industrial wastewaters.

5.8.5.2.2   Anaerobic Filter

The Anaerobic Filter as in Figure 5.45 (b) is a tank in which microbial cells are both entrapped as clumps of cells in the interstices between packing material and as biofilm attached to the surface of the packing material. The packing or filter media is usually of naturally crushed rock of 15 mm to 25 mm size or consisting of plastic or ceramic material. The filter media should have high specific surface and porosity to allow for maximum possible film growth and retention of biomass. The reactor is operated as up flow submerged packed bed reactor. A number of such filters have been constructed for treatment of low strength wastes such as municipal sewage.

5.8.5.2.3   Anaerobic Fixed Films Reactor

The Anaerobic Fixed Film Reactor as in Figure 5.45 (c) is a tank in which the microbial mass is immobilized on fixed surfaces in the reactor. It is operated in down flow mode to prevent accumulation of refractory particulates contained in the influent and sloughed biofilm. The sloughed biofilm is also discharged with the effluent. The reactor may be operated in either submerged or non-submerged condition. The reactor packing is usually of modular construction consisting of plastic sheets providing a high void ratio. Such reactors have been constructed to treat high strength wastes.

5.8.5.2.4   Fluidized and Expanded Bed Reactor

The Fluidized Bed reactor as in Figure 5.45 (d) is a tank which incorporates an up flow reactor partly filled with sand or a low density carrier such as coal or plastic beads. A very large surface area is provided by the carrier material for growth of biofilm. The system readily allows passage of particulates, which could plug a packed bed, but requires energy for fluidization. Expanded Bed (EB) reactors do not aim at complete fluidization and use a lower up flow velocity resulting in lesser energy requirement. These reactors can be used for treatment of municipal sewage as well.

5.8.5.2.5   Up flow Anaerobic Sludge Blanket Reactor

The Up flow Anaerobic Sludge Blanket Reactor (UASB), Figure 5.45 (e), maintains a high concentration of biomass through formation of highly settleable microbial aggregates. The sewage flows upwards through a layer of sludge. At the top of the reactor phase, separation between gas-solid-liquid takes place. Any biomass leaving the reaction zone is directly recirculated from the settling zone. The process is suitable for both soluble wastes and those containing particulate matter. The process has been used for treatment of municipal sewage at few locations and hence limited performance data and experience is available presently.

5.8.5.3   Design and Operational Considerations

Appendix A.5.15 presents an illustrative design of Upflow Anaerobic Sludge Blanket Reactor.

Appendix A.5.16 presents an illustrative design of Anaerobic Filter.

5.8.5.3.1   Organic Load and Sludge Retention Time

It is customary to express the organic matter in sewage in terms of Biochemical Oxygen Demand (BOD) or Chemical Oxygen Demand (COD). In anaerobic treatment systems, the COD value is finding greater usage, which lends itself directly to mass balance calculations. Reduction in COD for municipal sewage would normally correspond to equivalent amount of ultimate BOD reduction. Table 5.15 summarizes volumetric organic loads used in some of reactors for municipal sewage.

Table 5.15 Organic Loadings and Performance Efficiencies of Some High Rate Anaerobic Reactors

Reactor Type	Organic Load kg COD/m^{3}d	Efficiency %
AF	0.3 - 1.2	65 - 75
UASB	1.0 - 2.0	50 - 70

Source: CPHEEO, 1993

SRT, which is a more rational design parameter, is difficult to calculate for anaerobic reactors. For anaerobic contact and UASB, it ranges between 15 to 30 days while for other systems it is estimated to be about 100 days or more, giving them greater operational stability.

5.8.5.3.2   Hydraulic Load

For dilute wastes, the minimum HRT at average flow may be 6 to 12 hours for wastes containing suspended organic matter. In UASB reactors where a settling zone is provided, the average hydraulic over flow rate should not exceed 1 m/hour for flocculent sludge and 3 m/hour for granular sludge. The velocity through port between reaction zone and settling zone for the two types of sludge should not exceed 3 and 12 m/hour respectively. The face velocities depend on the characteristics of the media used.

5.8.5.3.3   Effect of Temperature

Activity of methanogenic bacteria is strongly influenced by temperature. It approximately doubles for every 10°C rise in temperature in the range of 18 to 38°C. However, high micro-organism concentration in high rate anaerobic reactor compensates the decreased activities of the anaerobic organisms at lower temperature.

5.8.5.3.4   Excess Sludge Production and Nutrient Requirement

In UASB treatment systems directly treating municipal sewage, the sludge production is reported to be 0.1 to 0.2 kg dry matter/m^3 sewage treated or 0.4 to 0.7 kg dry matter / kg BOD removed, and these include both inert matters present in raw sewage and end products of biological synthesis. The sludge production due to microbial synthesis from anaerobic systems is of the order of 0.01 to 0.1 kg VSS/kg COD removed. The lower values are for systems maintaining high SRT values. Consequently, the requirement of nitrogen and phosphorus is also low. In addition to nitrogen and phosphorus, methanogenic bacteria also require iron, cobalt, nickel and sulphide. These elements are usually present in sewage, but may have to be added if needed.

5.8.5.3.5   Toxicity

Anaerobic bacteria like most micro-organisms can be acclimated to different levels of various toxicants. However, because of their slow growth rate the acclimatization period may be comparatively longer. The sewage characteristics should be evaluated for their toxic effects before anaerobic treatment is adopted.

5.8.5.3.6   Recirculation

Recirculation may be practiced for dilution of incoming waste organic matter and/or biodegradable toxicants; it also provides flow for fluidization in case of FB/EB reactors. In case of municipal sewage, no recirculation is required except for fluidization in FB/EB reactors.

5.8.5.3.7   Gas Yield and Utilization

Methane production can be directly related to degree of treatment based on of COD value of methane produced and COD reduction. Theoretically, 0.35 m^3 methane is produced per kg COD reduction. Biogas normally contains 65% to 70% methane and 30% to 35% carbon dioxide. Since for low strength wastes there is considerable throughput of liquid in a high rate anaerobic treatment system, the gases also escape from the system with the effluent in soluble form. For municipal sewage, therefore, only 0.15 to 0.2 m^3 methane/kg COD removed may be recovered. Further, because of considerably higher solubility of carbon dioxide in comparison to methane, the off gas is enriched in its methane content to about 90%.

The generation of biogas is considered an asset of anaerobic sewage treatment. It is true for anaerobic digestion of sludge and strong industrial wastes where large amounts of gas may be generated. However, in the case of "weak" municipal sewage the recovery is less. Furthermore, for financial viability there should be an opportunity for utilization of the gas. Direct use of biogas in boiler houses in industries, utilization in institutions or in households is a more attractive option compared to generation of electricity which requires greater initial investment and operational and maintenance cost.

5.8.5.4   Pre-Treatment

Screening and grit removal are commonly used pre-treatment unit operations before direct anaerobic treatment.

5.8.5.5   Effluent Quality and Post Treatment

In the case of treatment of municipal sewage, the effluent BOD can be expected to be about 50 mg/l assuming influent BOD of 200 mg/l. For concentrated wastes, the BOD concentration would be higher. Depending on the situation, one or more of the following post-treatment operations may be considered:

i)   Holding pond of one day detention time followed by Fish pond/aqua culture pond

ii) Aerobic treatment (aerated lagoons, oxidation pond, etc.)

iii) Tertiary chemical treatment processes

5.8.5.6   Choice of Process

In general, anaerobic treatment does not produce the treated sewage quality fit for discharge for any of the receiving environments and invariably a downstream aerobic treatment is needed. Further, the generation of foul odours as sulphide and potential methane which can ignite and are relevant. These are the experiences with UASB plants adapted for sewage in India have also not been greatly enthusing. As such, the following recommendations are made out.

a) Where facultative ponds are proposed, anaerobic ponds may be used as pre-treatment to reduce the land area of the downstream facultative pond and if feasible the methane gas can be collected by a synthetic gas dome and used if necessary after sulphide stripping.

b) The use of UASB shall be discontinued gradually, over a period of time.

5.8.6   Supplemental Treatment Processes

5.8.6.1   Historical

Historically, the general objective of sewage treatment has been to achieve a BOD of 20 mg/l and SS of 30 mg/l. With the passage of time, the need to ensure against waterborne pathogen removal and against eutrophication, the recent STP tenders brought in the additional stipulations to control the eutrophication of receiving waters and ensuring better removal of waterborne pathogenic organisms in treated sewage, besides reducing the oil and grease limit to 5 mg/l. These requirements have necessitated   augmentations of conventional secondary treatment and technologies beyond the secondary treatment and referred to as the tertiary treatment. The technologies are dealt with below.

5.8.6.2   Tertiary Treatment Technologies

By definition, these are removal of constituents beyond the ability of secondary treatment. These are Chemical Precipitation and Membrane Technologies.

5.8.6.2.1   Chemical Precipitation

This is required to remove the phosphorous for control of eutrophication in receiving waters, salts if the treated sewage is to be used for industrial purposes and heavy metals. The technology part alone is dealt with here. The guidelines for these are dealt with in Chapter 7. The precipitation reactions in water are shown in Table 5.16 (overleaf) for an illustrative composition in water.

If we refer to the Table 5.16 there are five rows for each chemical to be removed. For example, if we consider the removal of magnesium sulphate, the first row names the chemical being considered, the second row presents the chemical reaction formula, the third row shows the stoichiometry, the fourth row shows the actual quantity of the chemical and the quantity of the reactant needed and the fifth row verifies the stoichiometry of the actual reaction.

The last rows sum up the weight of chemicals added and the weight of chemicals precipitated. These are theoretical equations and in actual practice, the minimum solubility of each chemical ion plays a part.

Table 5.16 Illustrative Chemical Water Hardness Precipitation Reactions and Chemical Needs

Precipitation Reactions of $MgSO_4 - CaSO_4$					
$MgSO_4$	+	$Ca(OH)_2$	=	$Mg(OH)_2$	+	$CaSO_4$
120		74	=	58	+	136
194			=	194		
2066	+	1274	=	999	+	2341
3340			=	3340		
Precipitation Reactions of $MgCl_2 - CaCl_2$						
$MgCl_2$	+	$Ca(OH)_2$	=	$Mg(OH)_2$	+	$CaCl_2$
95		74	=	58	+	111
169			=	169		
4253	+	3313	=	2597	+	4969
6410			=	6410		
Precipitation Reactions of $CaCl_2$						
$CaCl_2$	+	Na_2CO_3	=	$CaCO_3$	+	$2NaCl$
111	+	106	=	100	+	117
217			=	217		
4969	+	4745	=	4477	+	5237
9714			=	9714		
Precipitation Reactions of $CaSO_4 - Na_2SO_4$						
$CaSO_4$	+	Na_2CO_3	=	$CaCO_3$	+	Na_2SO_4
136		106	=	100	+	142
242			=	242		
2341+4313=6654	+	5186	=	4893	+	6947
11840			=	11840		
Precipitation Reactions of Na_2SO_4 – No further precipitations needed						
Na_2SO_4	+	$BaCl_2$	=	$BaSO_4$	+	$2NaCl$
142		208	=	233	+	117
350			=	350		
6947	+	10176	=	11399	+	5724
17123			=	17123		
Chemicals added						
$Ca(OH)_2$ = 1274 + 3313 = 4587 (+) Na_2CO_3 = 4745 + 5186 = 9931 (+) $BaCl_2$ = 10176 = 24694 mg/l						
Precipitates						
$CaCO_3$ = 4477 + 4893 = 9370 (+) $Mg(OH)_2$ = 999 + 2597= 3596 (+) $BaSO_4$ = 11399 (+) 92 SiO_2 = 24457 mg/l						

Mass balance becomes 24694 Vs 24457. This yields 99 % and is ok. If decimals are accounted for, there will be 100 % balance

For example, the Calcium and Magnesium can never be precipitated all the way down to zero and there will be a residual of 15 to 20 mg/l remaining as ion in solution. Moreover, the equations are pH dependant especially for Ca and Mg as shown in Figure 5.46 and Figure 5.47.
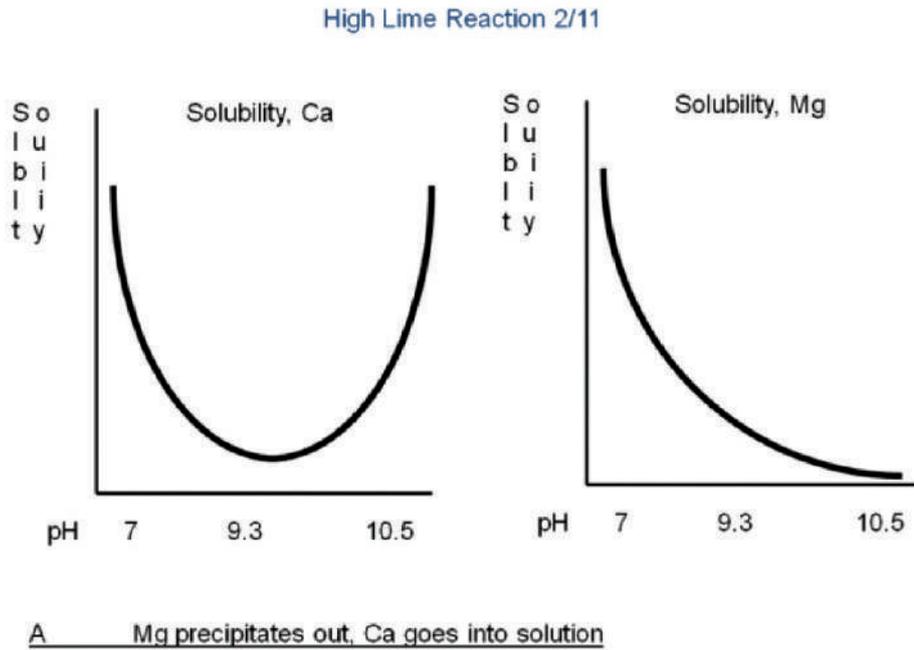
Figure 5.46 Calcium and magnesium solubility with pH variation in water
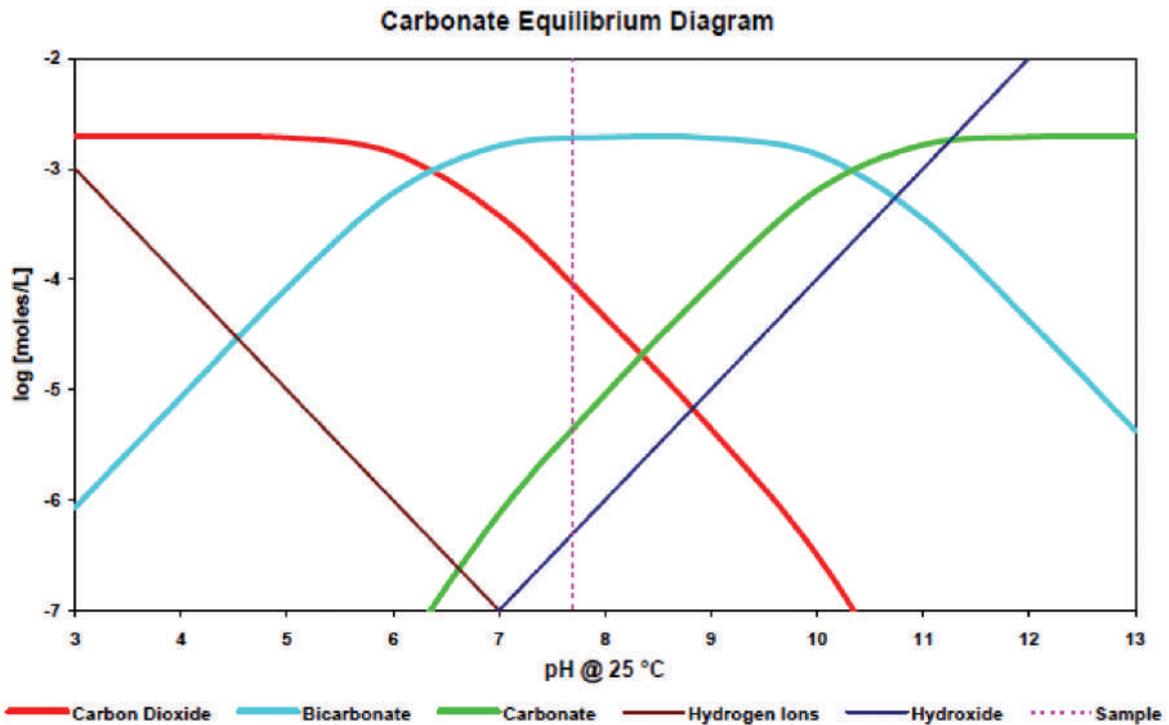
Figure 5.47 Calcium carbonate equilibrium at pH variations

An important factor to be considered in applying these equations to sewage is the fact that even a trace of phosphorous interferes with the precipitation of the compounds of reaction because it coats these compounds as a slimy layer similar to what occurs when Calcium Phosphate when used to prevent scale formation in cooling water circuits. Thus, in applying the equations of Table 5-16 to sewage, it needs to be stressed that the organic matter is to be first removed along with as much phosphorous in secondary treatment by augmenting the same and then only the hardness removal by chemical softening has to be thought off.

This again brings in the question of first removing the phosphorous before attempting to remove the hardness implying a two stage chemical softening. The chemical precipitation of phosphorous is by the use of Ferric or Aluminium salts.

For each Kg of phosphorous 0.9 kg of Aluminium or 1.8 kg of Iron is needed, showing that the sludge production is less by half by using Aluminium. The chemical equations are as under

$$Al_2(SO_4)_3 \cdot 18H_2O + 2H_3PO_4 = 2AlPO_4 + 3H_2SO_4 + 18H_2O \qquad (5.41)$$

$$FeCl_3 \cdot 6H_2O + H_2PO_4^- + 2HCO_3^- = FePO_4 + 3Cl^- + 2CO_2 + 8 H_2O \qquad (5.42)$$
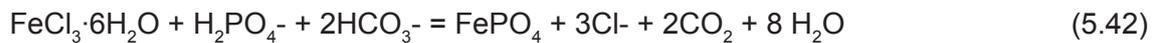
Figure 5-48 shows the phosphorus solubility with pH variation in water.
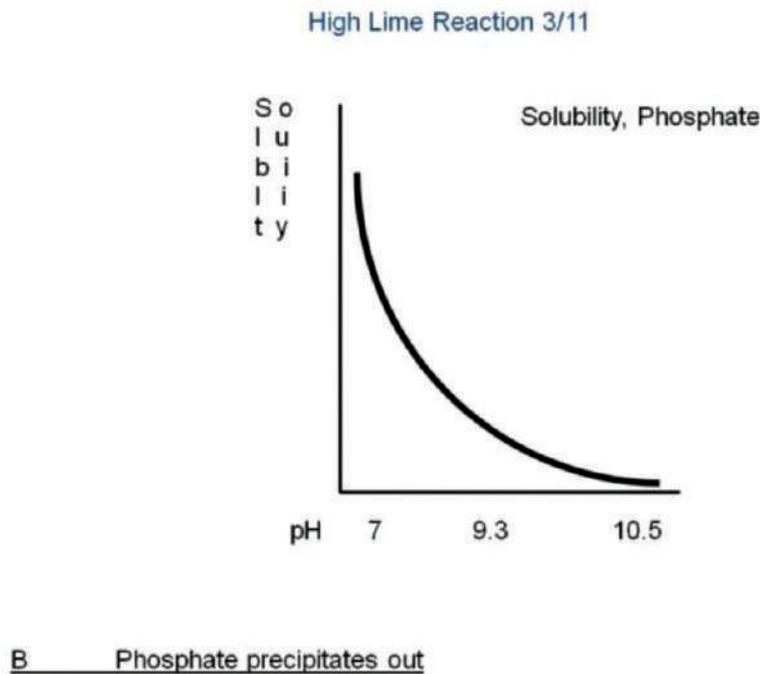
Figure 5.48 Phosphorus solubility with pH variation in water

There are different locations for the addition of the chemical as per various authors. It is added either in the primary clarifier or the aeration tank or in the tertiary stage as in Figure 5.49 (overleaf). In actual practice a separate tertiary stage gives more flexibility and control.
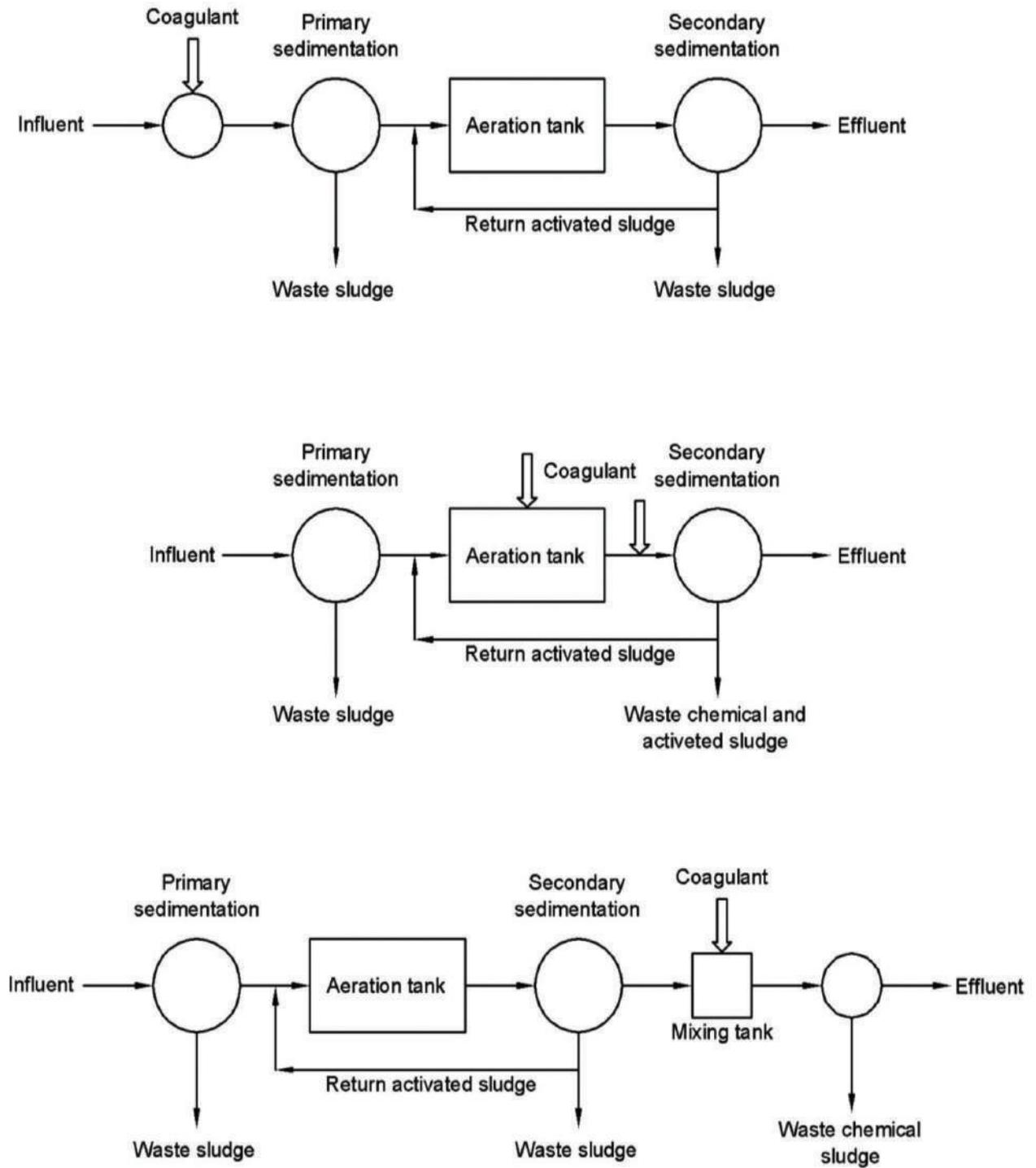
Figure 5.49 Optional points of addition of coagulant chemical in STP
for phosphorus precipitation

Another technology is the high Lime followed by carbonation or acidification. In this case, Lime is added to increase the pH to above 10.5, whereby the Magnesium and Phosphorous are precipitated and simultaneously Silica is co-precipitated with Magnesium at 1:5 as ions. At the same time, by providing a contact period of minimum 45 minutes the waterborne pathogenic organisms are rendered immobile by a process of solidifying their protoplasm thus killing them. In addition, the trace metals present in the aqueous medium are precipitated as their oxides, as the pH increases and once precipitated, they cannot go back into solution. After settling out, pH of the supernatant is carbonated by diffusing carbon dioxide gas. Two-stage carbonation is preferred. In the first stage, the pH is kept around 9.3 to bring about maximum precipitation of the dissolved Calcium as the Calcium carbonate. The pH of the settled overflow is then reduced to around 7 in a separate second stage to dissolve all Calcium as Calcium bicarbonate. The pH can also be reduced by acidifying, but this will convert the Calcium to its sulphate or chloride and increase the TDS.

This high Lime and carbonation technology was evolved many decades back and is very useful in industrial reuse of treated sewage for cooling purposes.

5.8.6.2.2   Membrane Technologies

These are the alternative to removal of hardness, but they result in the removed salts as reject solution, which will need either an ocean disposal or thermal evaporation. Besides they require extensive pre-treatment to eliminate suspended solids altogether. The design guidelines for these are presented in Section 5.18.10.3.

5.9   DISINFECTION FACILITIES

5.9.1   Need for Disinfection

Disinfection of treated sewage may be needed when the receiving water quality may be affected by the Coliforms after the discharge. A case in point is the documentary of the fate of Coliform organisms in India's most revered and most used river Ganga. The habitations supported by this river basin and its tributaries are shown in Figure 5-50 (overleaf) . The results in Table 5-17 shows the BOD and D O at the origin, en route and final confluence over a traverse of 2525 km covering 23% of the country and supporting 43% of its population. Recognizing the seriousness of the pollution, the Ganga Action Plan was completed between the years 1993 and 2000 for sewering and, STPs for 53 habitations covering 1000 MLD. The impact was surely felt, but was not complete enough as can be seen from Table 5.2 read along with Table 5.18.

The point of concern is that despite such massive implementation of sewerage and sewage treatment, almost all stretches elude their up-gradation to the desired grade of at least C, for the rest of the 2100 km after the initial 400 km.

A more recent ongoing work by the IIT's reveals the rather high levels of Total coliforms and Faecal coliforms along this entire course as in Figure 5.51 and Figure 5.52.

The issues of concern are to be read along with Table 5-17, Table 5.2 and Table 5.18 which correlates the total coliform limits for the river classifications A to E.
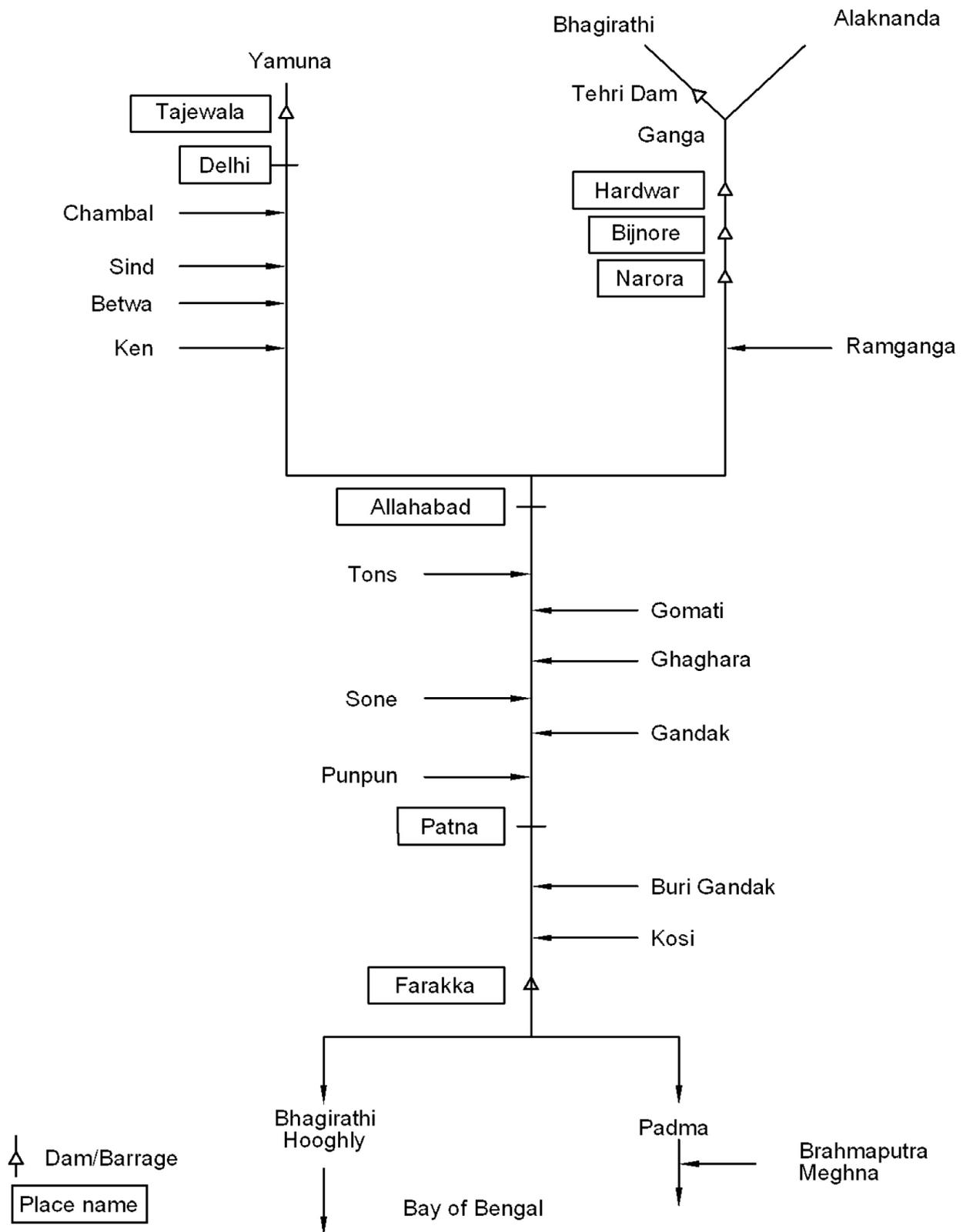
Source: MOEF, 2009

Figure 5.50 The Traverse of Ganga, the Country's most revered and longest river.

Table 5.17 Variation of data of Ganga River during 1986-2008

No	Location	km	1986		1993		2002		2005		2008		Standard Values	
			(a)	(b)	(a)	(b)	(a)	(b)	(a)	(b)	(a)	(b)	(a)	(b)
1	Rishikesh	0	8.1	1.7	9.0	1.3	8.2	1.2	8.5	1.0	8.1	1.2	5.0	3.0
2	Haridwar	30	8.1	1.8	7.2	1.4	7.8	1.7	8.1	1.4	7.9	1.4	5.0	3.0
3	Garhmukteshwar	175	7.8	2.2	8.5	1.6	7.5	2.1	7.8	2.0	7.8	1.9	5.0	3.0
4	Kannauj U/S	430	7.2	5.5	7.2	2.3	7.7	1.2	8.5	1.7	6.5	2.9	5.0	3.0
5	Kannauj D/S	433	6.5	5.1	8.4	2.5	6.5	4.2	7.6	4.5	6.2	3.1	5.0	3.0
6	Kanpur U/S	530	7.2	7.2	7.5	1.9	6.3	3.8	6.2	4.3	4.9	3.4	5.0	3.0
7	Kanpur D/S	548	6.7	8.6	5.2	24.5	6.7	4.9	4.7	5.4	6.0	4.1	5.0	3.0
8	Allahabad U/S	733	6.4	11.4	6.9	1.8	13.0	8.0	8.5	5.5	8.4	4.8	5.0	3.0
9	Allahabad D/S	743	6.6	15.5	7.2	1.9	8.2	3.8	8.4	3.1	7.7	3.2	5.0	3.0
10	Varanasi U/S	908	5.6	10.1	8.2	0.8	10.8	3.0	8.6	2.0	7.5	2.2	5.0	3.0
11	Varanasi D/S	916	5.9	10.6	7.6	1.0	7.5	2.5	8.3	2.3	7.3	3.0	5.0	3.0
12	Patna U/S	1188	8.4	2.0	8.2	1.2	7.1	1.9	7.4	2.0	6.0	1.7	5.0	3.0
13	Patna D/S	1198	8.1	2.2	8.0	1.5	7.1	2.0	8.0	2.2	5.9	2.4	5.0	3.0
14	Rajmahal	1508	7.8	1.8	8.5	0.7	7.9	1.5	7.4	1.8	6.2	2.0	5.0	3.0
15	Palta	2050	7.3	1.0	7.1	0.9	7.3	2.7	7.0	3.0	6.9	2.2	5.0	3.0
16	Uluberia	2500	5.8	1.1	6.1	0.9	5.4	1.9	5.4	2.6	5.3	3.6	5.0	3.0

(a) = DO in mg/l, (b) = BOD in mg / l

Source: MOEF, 2009

Table 5.18 Classification of Ganga water at various locations according to designated best use

Locations	Desired Class	Observed Class					
		1997	1998	1999	2000	2001	2008*
Rishikesh	A	D	B	C	NA	C	B
Haridwar	B	C	C	C	NA	C	B
Garhmukteshwar	B	B		D	NA	D	NA
Kannauj U/S	B	D	D	D	D	D	D
Kannauj D/S	B	D	D	D	D	D	D
Kanpur D/S	B	D	NA	D	D	D	D
Raibarreilly	B	D	D	D	NA	NA	NA
Allahabad U/S	B	D	E	D	NA	NA	C
Allahabad D/S	B	D	E	D	NA	NA	D
Varanasi U/S	B	D	D	D		D	D
Varanasi D/S	E	E	E	E	NA	NA	D
Gazipur	B	D	D	D	D	NA	NA
Buxar	B	D	D	D	D	D	C
Patna U/S	B	D	D	D	D	D	NA
Patna D/S	B	D	D	D	D	D	NA
Rajmahal	B	D	D	D	D	D	D
Palta	B	D	B	NA	D	D	D
Uluberia	B	D	B	NA	D	D	D
Locations		Critical Parameter					
		1997	1998	1999	2000	2001	2008*
Rishikesh		CF	CF	CF			
Haridwar		CF	CF	CF			
Garhmukteshwar		BOD	BOD			BOD,CF	
Kannauj U/S		BOD,CF	BOD	CF	CF	CF	CF
Kannauj D/S		BOD,CF	BOD	CF	BOD,CF	BOD,CF	BOD,CF
Kanpur D/S		BOD,CF		CF	CF	CF	CF
Raibarreilly		CF	CF	CF			
Allahabad U/S		BOD,CF	CF	CF			CF,BOD
Allahabad D/S		BOD,CF	CF	CF			CF,BOD
Varanasi U/S		BOD,CF	BOD	CF		CF	BOD,CF
Varanasi D/S		DO,BOD,CF	DO,BOD	BOD			BOD,CF
Gazipur		BOD,CF	BOD	BOD	CF		
Buxar		BOD	CF	CF	CF	CF	CF
Patna U/S		CF	CF	CF	CF	CF	
Patna D/S		CF	CF	CF	CF	CF	
Rajmahal		CF	CF	CF	CF	CF	CF
Palta		BOD			BOD,CF	BOD,CF	CF
Uluberia						BOD,CF	BOD,CF

A- Drinking Water Source without conventional treatment but after disinfection
B- Outdoor bathing (Organized)
C- Drinking Water Source with Conventional treatment followed by disinfection
D- Propagation of wild life and fisheries
E- Irrigation, industrial cooling and controlled waste disposal
CF-Coliform Indicator Organisms
Source: MOEF, 2009
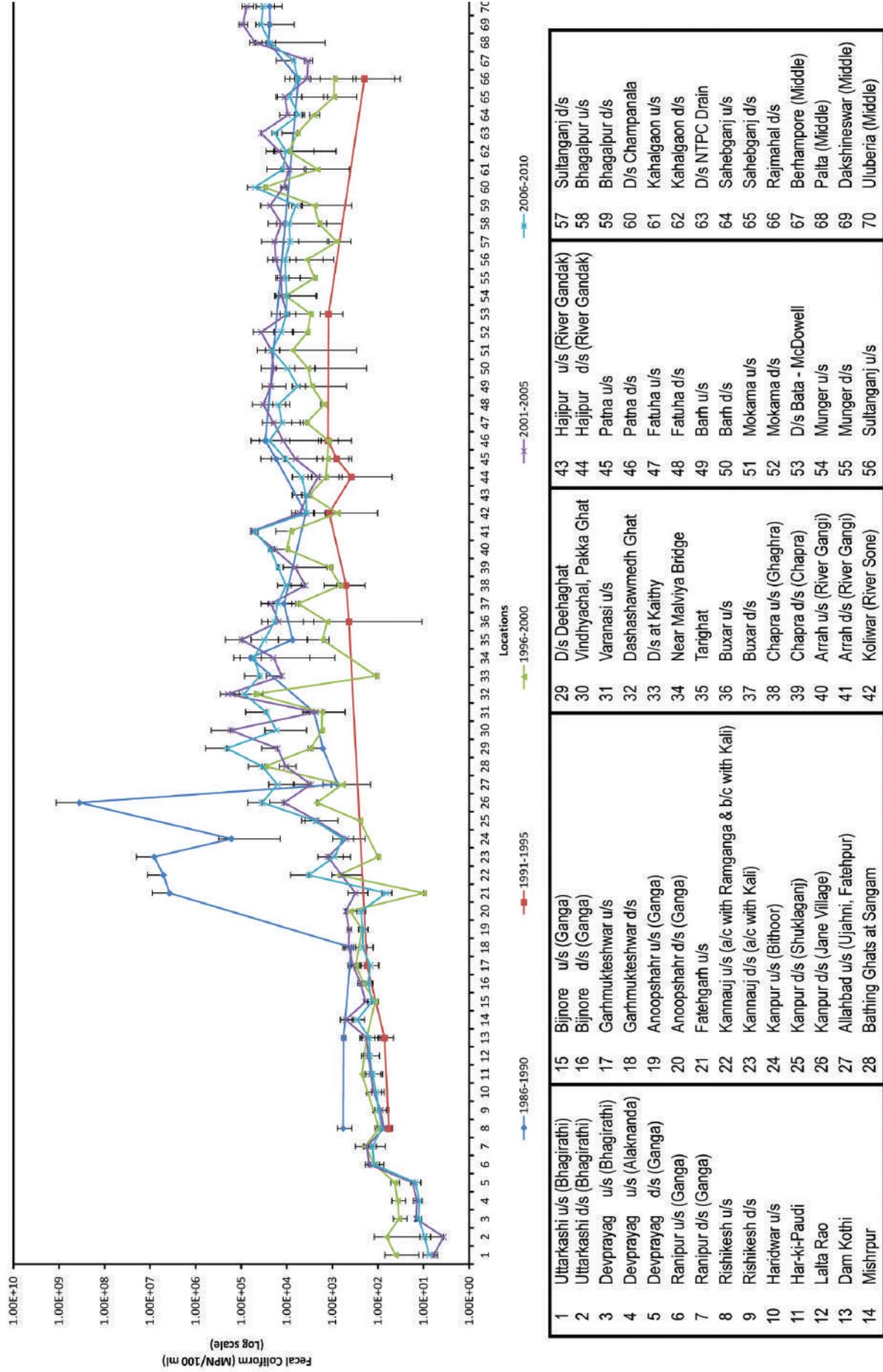
Figure 5.51 Variation in 5-year average Faecal Coliform at various locations along the Ganga River
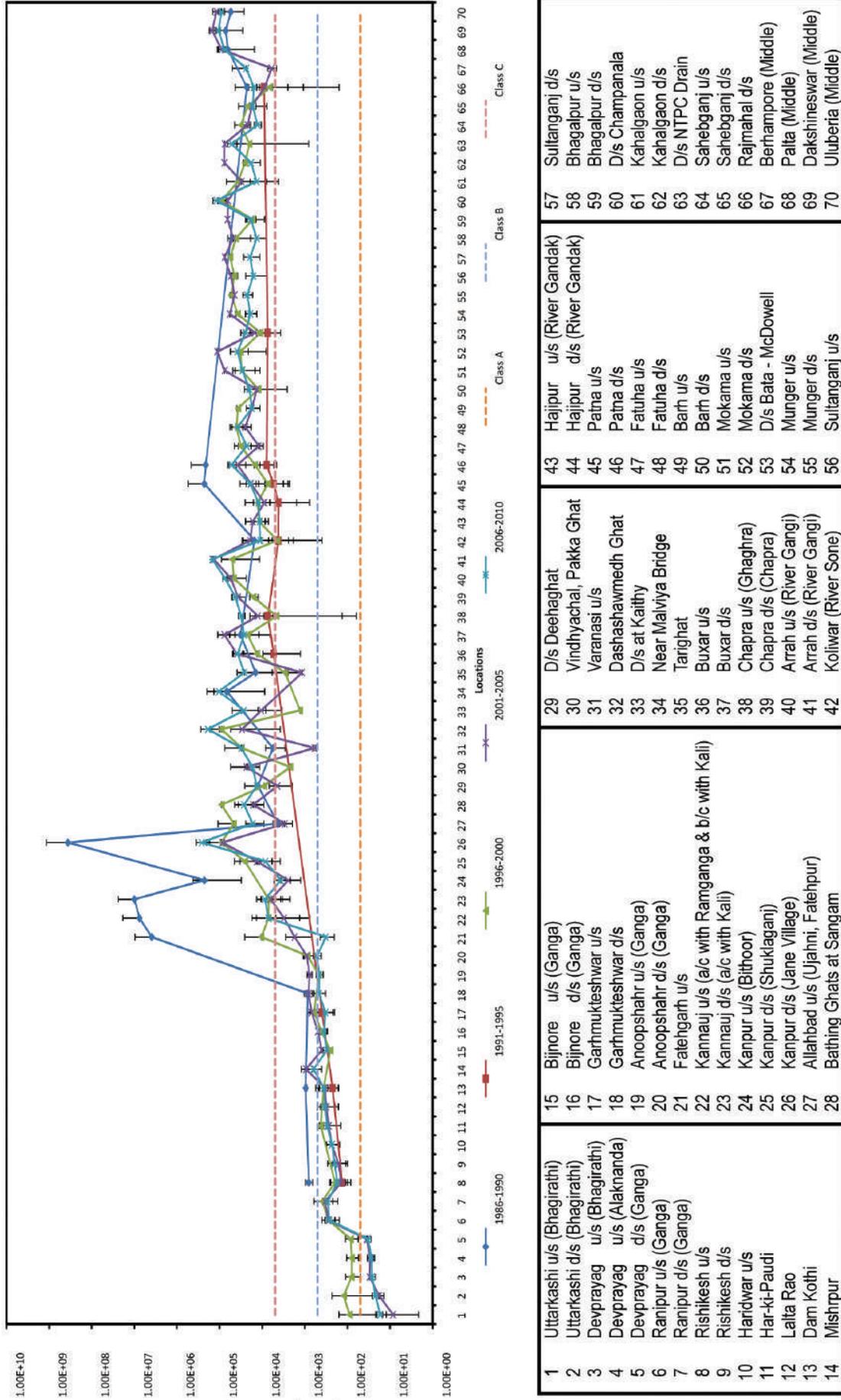
Figure 5.52 Variation in 5-year average Total Coliform at various locations along the Ganga River

Thus, it is to be conceded that once the initial 400 km is traversed, almost the entire remaining stretch is not fit to the desired classification of at least C water quality. Here is the realistic need for disinfection of the treated sewage discharges to reverse the trend. Coliform organisms are indicator organisms for presence of waterborne disease causing organisms like potentially epidemic causing Cholera, Typhoid, Jaundice, etc. The faecal coliforms are a sub-group within the total coliforms and are a reinforcing indicator-organisms. The present situation does not portend well for the future of this river. The origins of faecal coliforms can be only from human activities of improper sewerage and inadequate sanitation. As such, there arises a need to look into more of coliform aspects and extend beyond the typical discharge standards of 20 mg/l BOD and 30 mg/l of SS when specifying discharge standards for treated sewage discharging into surface waters, which are to be used as potable water sources. Reference to Appendix A.2.1 also needs to be recalled in this context of faecal coliforms. The position in other non-perennial water-courses in the country cannot be anything better.

Recent tenders for STPs are already restricting the Faecal Coliform limit at not over 200/100 ml but the raw sewage value is in the range of 16,00,000/100 ml. As earlier shown in Table 5.5, the reduction even in ASP is only about 96%, which means the treated sewage will have 64,000/100 ml, in order to kill the same and attain the limitation, disinfection of the treated sewage is needed.

5.9.2   Disinfection by Chlorination

This is the most widely used technology in both water supply and sewage treatment. As the treated sewage is fresh from secondary aerobic biological treatment, the chlorination of such effluents does not result in hazards. In the case of effluents from anaerobic processes like UASB, the provision of an aerobic polishing treatment is mandatory before chlorination. The usual dosage used is 10 mg/l and the flow through detention time in the contact tank is 30 minutes based on average flow. Suitable baffles are provided in these tanks to maximize the contact. These tanks shall not be covered. This is because the chlorine gas may permeate into the concrete in the roof and corrode the concrete slab. There is no way this can be detected periodically. It will be known only when the roof collapses. The worst situation will be when operators are standing on the roof. Hence, open tanks and free wind movement must be allowed to blow across the tank. This will also help in detecting excess chlorination. The residual chlorine after the contact has been generally detected at 1 to 1.5 mg/l at the maximum and there are no offensive odours arising there from. In actual practice, the dosage can be varied to be in conformity with faecal coliform limits.

5.9.2.1   De-chlorination

Excess of residual chlorine if any is nullified by dechlorination chemicals like sulphur dioxide (SO_2) gas or salts as sodium thiosulfate ($Na_2S_2O_3$), sodium sulphite (Na_2SO_3), sodium bisulfite ($NaHSO_3$), sodium   metabisulfite ($Na_2S_2O_5$), calcium thiosulfate (CaS_2O_3), ascorbic acid (Vitamin C) and sodium ascorbate. sodium bisulfite is used by some utilities due to its lower cost and higher rate of de-chlorination. Sodium sulphite tablets are chosen by utilities due to ease of storage and handling, and its ease of use for de-chlorinating constant, low flow rate releases. Sodium thiosulfate is used for de-chlorination since it is less hazardous in handling and consumes less oxygen than sodium bisulfite and sodium sulphite. Ascorbic acid and sodium ascorbate are used because they do not impact the D O concentrations. Several chemicals are available for de-chlorination.

Additionally, chemicals such as Sodium Metabisulfite, Sodium Sulphite and Sodium Thiosulfate may deplete D O of receiving streams under certain circumstances. Sodium Metabisulfite and Ascorbic acid may decrease the pH of some waters. It is necessary to determine in the laboratory the choice of the chemical for the given sewage quality and keep stock of the chemical for a demand of at least a week. The standard engineering procedures for dispensation shall be organized in consultation with the Material Safety Data Sheet (MSDS) and the recommendations of the authorised supplier.

5.9.3   Ultraviolet Radiation Disinfection

Ultraviolet rays are most commonly produced by a low pressure mercury lamp constructed of quartz or special glass which is transparent and produces a narrow band of radiation energy at 2537 A0 emitted by the mercury vapour etc. Though this is a standard chemistry, in actual practice, its efficiency is largely constrained by the requirements of (a) The water to be free from suspended and colloidal substances causing turbidity, (b) The water does not contain light absorbing substances such as phenols, ABS and other aromatic compounds, (c) The water is flowing in thin film sheets and is well mixed and (d) Adequate intensity and time of exposure of UV rays. The advantage of UV is that exposure is only for short periods, no foreign matter is actually introduced and no toxic and no odour is produced. Over exposure does not result in any harmful effects. The disadvantages are that no residual effect is available and there is lack of field test for assessing the treatment efficiency. Moreover, the equipment needed is expensive.

5.9.4   Ozone Systems

It is a faintly blue gas of pungent odour. Being unstable, it breaks down to normal oxygen and nascent oxygen. This nascent oxygen is a powerful oxidizing agent and germicidal agent. Ozone is produced by the corona discharge of high voltage into dry air and being unstable has to be produced on-site. It poses more superior bactericidal properties than chlorine and is highly effective in removal of tastes, odours, iron and manganese. As Ozone reacts with chemical impurities prior to attacking the microorganisms, it produces essentially no disinfectant unless ozone demand of water has been satisfied, but much more rapid kills are achieved once free ozone residuals are available. Studies have reported 99.99% kills of E Coliform within less than 100 seconds in the presence of only 10 mg/l of free residual ozone. Moreover, the efficiency of its disinfection is unaffected by pH or temperature of the water over a wide range. Among the disadvantages are (a) its high cost of production, (b) its inability to provide residual protection against recontamination and (c) the compulsion for its generation on-site due to instability.

5.9.5   Relative Aspects of Disinfection Processes

In a recent finding the US Water Environment Federation (WEF) observed that "disinfection of wastewater protects the public from potential exposure to pathogenic microorganisms that would otherwise be present in wastewater effluent that is discharged into water bodies that may be used for recreation or drinking water. Wastewater disinfection has traditionally been accomplished using some form of chlorination. In fact, more than 60% of the 20,000 municipal wastewater treatment plants in North America use chlorination as the primary method of disinfecting effluent. Although an effective disinfectant, chlorine (and related compounds) has come under increased scrutiny because of regulatory, safety and security issues.

An on-going unpublished work observes that after studying the performance of disinfectants of chlorination, its variants, solar, UV, Ozone and Peracetic acid, the faecal coliform removal was about the same at up to 4<5 log unit, except in the case of solar where it was up to 2<3 and total coliforms <1000 for all except UV at <100 and Ozone at < 50.

The occurrence and fate of disinfection by-products and related residuals are not readily available in validated reportings.

The relative efficiencies of disinfectants vs. their by-products is long engaging the attention worldwide. Most of the reported works are only in respect of surface waters, ground waters, surface runoff waters, etc. The findings of these studies do not fully apply to disinfection of treated sewage. The US-EPA-Design Manual on Municipal Wastewater Disinfection-EPA/625/1-86/021 observes that even otherwise, the issue of attention has been the disinfection by-products. Though it is contended that chlorination may result in by-products of Trihalomethanes, it needs to be realized that it is the case only when chlorination of humic substances takes place and a treated sewage from an aerobic STP does not have humic substances. Moreover, the inherent alkalinity in sewage curtails on the THM formation potential because the alkalinity in sewage scavenges any hydroxyl free radicals.

In respect of UV, the distribution of biologically stable water is realized by reducing the AOC concentration using GAC filtration only after UV disinfection and as such, UV by itself is not a complete treatment. In respect of Ozonation, the overall effect of ozone on effluent toxicity have found the effects to be variable as ozonation of secondary sewage can both decrease or increase effluent toxicity. Considerable research is still needed on the formation of ozone by-products and the effect of ozone on the treated sewage. It also proposes an approach as in Table 5.19.

Table 5.19 Technical factors and feasibility considerations in disinfectant choice

No.	Considerations	Chlorine	Chlorine Dioxide	Ozone	UV
1	Flexibility	2	2	2	2
2	Reliability	1	2	3	2
3	Complexity	2	3	3	2
4	Effectiveness	2	1	1	2
5	Need for Piloting	1	4	3	3
	Rating based on scale of 1 to 5 with 1 indicating best degree of confidence				

Source: USEPA-EPA/625/1-86/021

In considering the foregoing in this sub-section, the recommendation of the CPCB in its publication titled (Performance of Sewage Treatment Plants - Coliform Reduction - CUPS/ 69 /2008-CPCB) that "one of the best methods of achieving 100% faecal microbes removal is coagulation-flocculation followed by chlorination after secondary treatment" appears appropriate as of now, pending more conclusive publications on other disinfection methods.

5.10   THE ISSUES OF NITROGEN AND PHOSPHORUS

It is to be recognized that large volumes of sewage, which far exceed the treatment capacity of sewerage systems and the natural capacity of rivers to purify water, are discharged untreated into rivers untreated. There is thus a serious degradation of the quality of river water, as well as deterioration of the living conditions in urban areas (Source : JICA Press Release-NR/2007 2 April 2, 2007). An example is the Agra city-water supply project, where ammonia pollution of the Yamuna river water is attributable to partly treated sewage discharged into it. The proposed treatment is a biological MBBR and ultra filtration membrane system for biological nitrification-denitrification of the raw water. By instituting this biological nitrification-denitrification in STPs itself, it is relatively easier to deal with far less volumes as compared to higher volumes of polluted water in such WTPs.

The nitrogen in sewage consists of organic nitrogen and ammoniacal nitrogen and their sum total is expressed as Total Kjeldahl nitrogen (TKN). The phosphorous is consisting of dissolved phosphorus and total phosphorus. The discharge standards permit the TKN at 100 mg/l and dissolved phosphorous at 5 mg/l. These chemicals are well known to contribute to eutrophication in receiving waters, especially stagnant ones.

There has been at least one instance in the country in which the uncontrolled sewage nutrients got accumulated in a downstream impoundment and the drought of that year resulted in high algal growth. The conventional water treatment plant with coagulation, sand filtration and disinfection could not remove the algal stench and the colour and the WTP was rejected outright by the population of Hosur town and water famine loomed large. The ground water was also at least 100 m deep. Emergency treatment was carried out for removal of nitrogen and phosphorous and algae by high lime and ammonia stripping followed by carbonation and sand filtration with final chlorination. The WTP was retrieved and the population was spared of a water chaos. (Source: S Saktheeswaran, 2011) The eutrophication issue has not assumed such proportions in the other water courses of the country.

However, if biological nitrification-denitrification can be done in STPs, such difficulties will not recur in the receiving waters.

Biological removal of phosphorous has undergone many advances since the days of Professor James L Barnard, the father of biological phosphorous removal. It is prudent to adopt such biological treatments in the STPs along with the traditional BOD removal.

Some of the biological STPs built in India have already established such a performance like the Koramangala and Chellagatta (K&C) valley STP at Bengaluru, constructed during 2004 under JICA assistance and some of the recent SBR plants.

5.11   DESIRABLE TREATED SEWAGE QUALITY AND PROCESSES

Considering the foregoing aspects and taking a comprehensive view, it appears necessary to embark on a set of tougher limitations of BOD, nitrogen and phosphorous especially in respect of treated sewage discharges into water bodies, which becomes a raw water source for drinking water projects of downstream habitations.

Kazmi et.al. after studies in respect of removal of coliforms in full-scale activated sludge plants operating in northern regions of India, concluded that the inter-relationship of BOD and SS with coliforms manifest.

The improvement of the microbiological quality of sewage could be linked with the removal of SS, and therefore, SS can serve as a regulatory tool in the absence of an explicit coliforms standard. (Kazmi et al., 2008).

Hence, it is recommended that the stringent limitations shall be henceforth adopted as guidelines for treated sewage discharge into surface water which after some travel may join a drinking water source to be used as source of supply for drinking as given in following Table 5.20

Table 5.20 Recommended Guidelines for Treated Sewage if Discharged into
Surface Water to be used as source of Drinking Water

Parameter	MOEF Standards (A)	Recommended Values
BOD, mg/L	30	Less than 10
SS, mg/L	100	Less than 10
TN, mg/L	100	Less than 10
Dissolved P, mg/L	5	Less than 2
Faecal Coliforms, MPN/100 mL	Not specified	Less than 230

(A) General Standards, Environmental Protection Rule, 1986 & as authorised by PCB

In order to achieve this, the treatment process would need to be designed for nutrient removal in addition to the conventional BOD and SS removal.

It has also been reported that if the nutrients were removed to the levels mentioned in Table 5.20, then the amount of chlorine required for disinfection would be less at about 5 mg/l.

However, the central and local regulatory agencies / bodies can set the discharge standards as felt appropriate.

Disinfection as in Section 5.9 will be required to achieve the Faecal Coliform (FC) limit. The following treatment processes shown in Figure 5.53 (overleaf) could be used for achieving nutrient removal.

If effluent BOD and SS of less than 10 mg/L cannot be achieved by biological processes, then they shall be followed by chemically enhanced settling process. The option of primary clarification and anaerobic digestion of sludge and gas recovery can also be considered based on capacity of plant and actual design values.

It also needs to be mentioned that discharge of treated sewage and untreated sewage mixing is to be totally avoided and hence interceptor sewers for transportation of the untreated sewage up to the STP should be practiced.
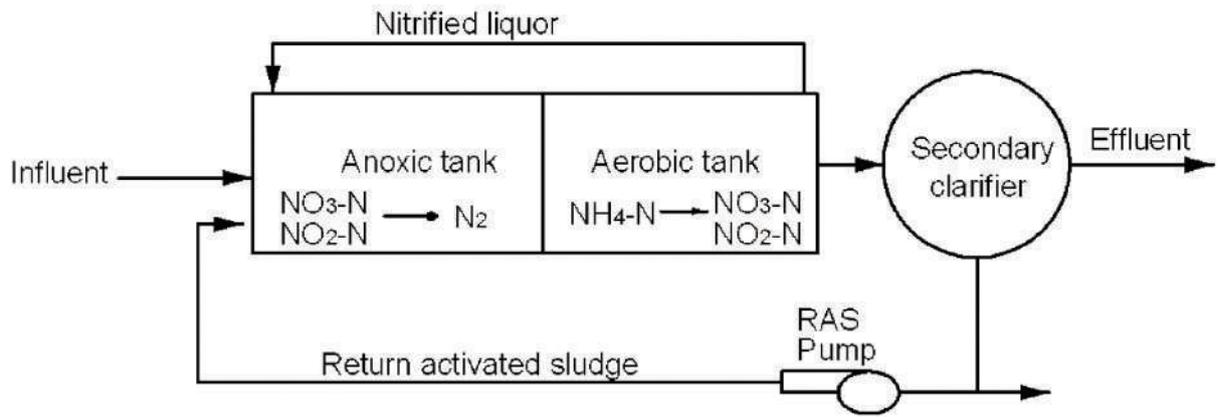
Case 1: Modified Ludzack Ettinger (MLE) Process with chemical enhanced settling & Chlorination
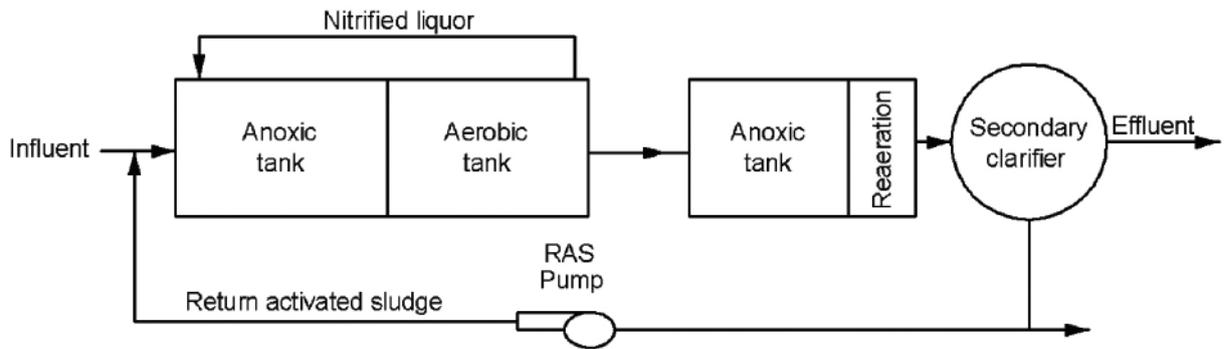
Case 2: Bardenpho process with Chlorination
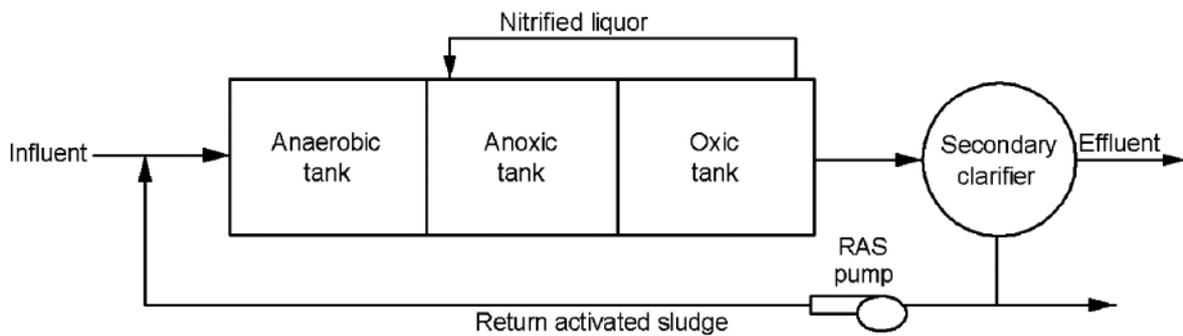
Case 3: Bardenpho process with Chlorination

Figure 5.53 Nutrient removal processes

5.12   ELECTRICAL AND INSTRUMENTATION

5.12.1   General

The electrical and instrumentation system in a sewage pumping station or sewage treatment plant is the same as any other system in other infrastructure projects. However, in addition, the following must be looked into.

a) Corrosive gas: Hydrogen Sulphide is a corrosive gas, which may be present in the air in some locations like sewage wells, pump dry pits and many of the treatment plant units. This gas can be more corrosive when humidity is very high like in coastal zones where it can be high. These gases can attack and damage the electrical contacts, in conductors and switches.

b) Inflammable gas: Methane is an inflammable gas, which may be present in the air in many units of the sewage pumping stations and treatment plants.

c) Variations of raw sewage flows: This causes the motors and switches to start and stop frequently, more than the recommended operations.

These needs, along with other general requirements of an electrical system have to be satisfied while deciding on the rating, sizing and designing of the electrical and instrumentation system.

These can be broadly classified into the following groups;

i)   Power receiving and transforming equipment (Substation & Transformers)

ii)   Motors and motor controllers (Starters, Cabling, etc.)

iii) Generators for captive / back-up power supply

iv) Uninterruptible power supply for protection and control circuits

v)   Instrumentation facilities

vi) Supervisory control and data acquisition system (SCADA)

5.12.2   Power Receiving and Transformation Equipment

Indoor / Outdoor substations are to be provided for higher voltage transformers in the upstream of indoor switchgear. A typical substation has the following major units.

1. Supply System:

    a) Low Tension Supply up to 150 HP (50Hz - 3Phase - 433V between phases)

    b) High Tension Supply above 150 HP through Indoor/Outdoor Substation (50 Hz-3Phase-11 KV/22 KV/33 KV in between phases)

2.   Lightning arresters

3.   Group operated switches (GOS) on both sides of the Circuit Breaker for 1000 KVA and above

4.   Circuit Breakers

5. Drop out fuses for outdoor substations

6. Overhead bus bars, conductors and insulators

7. Transformers

8. Current transformer and potential transformer for power measurement

9. Current transformer and potential transformers for protection in substations of above 1000 KVA

10. Fencing and Yard-lighting

11. Earthing

5.12.2.1 Location

The Load switch/Substation is normally located at one end of the plant, preferably in the vicinity of the largest load centre in the plant and the easy and unhindered accessibility of the power supply agency should be considered. An overhead line or underground cable from the LT or HT supply system of the power supply company may have to be brought and terminated at the Load switch/station. In the case of a sewage treatment plant, normally the raw sewage pumping station is usually the largest power centre, followed by the aerators/air compressors and recirculation pumps. The substation location is to be at an elevated place to avoid any flooding risks. Due consideration shall be given to easy entry and easy exit for vehicles at all times.

5.12.2.2 Rating and Supply Voltage

The rating of transformers is to be worked out by summing up all electrical loads that would work at a time, and adding a margin to it. The transformer is specified to work at its peak efficiency at about 75% of its rating so that a cushion is always available for any expansion, temporary loads, starting loads, and de-rating factors like possibility of higher ambient temperature, altitude, etc. The supply voltage of the substation will be decided by the total load current and the nearest supply voltage available.

5.12.2.3 Lightning Arresters

These are to be provided at the commencement of a substation conductor to draw away any power spike due to atmospheric and switching surges, and to protect the downstream equipment. The lightning arresters are earthed to conduct away the surge currents to the earth via earthing conductors and through earth pits.

5.12.2.4 Group Operating Switches (GOS)

These are to be provided as isolating facility to major equipment, such as a transformer or HT switchgear and come in different versions like single throw, double throw, earthed and unearthed for different applications and at a safe clearance height from the ground. These are made up from three separate switch units for each of the 3 phases, adding a common operating lever/rod, for operation from ground level. They are suitable for opening only when there is no load on the downstream side. In short, the downstream major loads should be switched off before switching off the supply by a GOS and the GOS should be locked in the open position to enable repair works to be done safely.

5.12.2.5   Circuit Breaker (CB)

These are insisted as an isolating mechanism for stations with transformers above 750 kVA by the power supply/Inspectorate authorities. However, it is common to provide Circuit Breakers even for 500 kVA transformer substations in the interest of a safe and reliable operation. The main difference between a GOS and CB is that, while GOS can be operated in the no-load condition only, a CB can be operated when the station load is on and more importantly, the CB has provision for tripping through a remote signal. This remote signal is useful for switching off supply when a fault or abnormal condition is sensed by the electrical metering / protection equipment. For small stations, there will be a single CB as main switchgear and one or more transformers will have GOS. For large substations there can be a main and additional transformer control CBs too. The sequence of operation of the CBs in such a case will be decided by suitably grading the currents at which the switchgear will trip. This is done at the protective relay taps. The tripping time of CBs is very short and is specified in milliseconds and these are of several types based on the insulating medium such as oil, vacuum, SF-6 gas, etc. Further, they are also classified on the voltage class as 3.6 kV, 7.2 kV, 12 kV, 24 kV, 36 kV, 72.4 kV, 123 kV etc.

5.12.2.6   Drop Out Fuses (DOF)

These are a means of protection to a fault current for small transformers up to 500 kVA. They are to be suspended from the pole and conductor connecting to the transformer primary. When the current exceeds a specified limit, the fuse wire in it melts and disconnects the supply and at the same time dropping off indicating the fault. They are normally connected just before the transformer primary connection. The fuse wires are made of an alloy of lead and tin and with inverse time characteristics. This means that the higher the short-circuit current, the faster will be the fuse blow-out time.

5.12.2.7   Overhead Bus Bars

These are conductors to carry power from each component to the next component in the power circuit in an outdoor station. They are to be rated to carry the maximum rated current continuously and the short-circuit current for a short time without damage.

5.12.2.8   Transformer

This is the most important component in the substation. It receives the electrical power at a higher voltage and steps it down to a lower service voltage. The transformers have insulating oil in their tanks where the HT and LT coils are wound around a core. The oil serves as an insulating and a cooling medium to disperse the heat that is generated during the operation. The oil is in turn cooled by circulating air around radiator fins. The connections on HT and LT side shall be through overhead bushings, cable termination or a combination of these. Being a large equipment and not readily available, it is preferable to have two transformers instead of one in a plant substation. For a standard ASP of conventional design, the normal power consumption is as in Table 5.21 overleaf.

There are essentially two types of transformers, Oil and Cast Resin. The oil cooled transformers are covered under IS: 2026 & IEC 60076. The cast resin transformers are covered under IS: 11171. It is an improved version compared to the oil cooled transformers up to 1600 kVA capacity. They are capable to withstand mechanical and thermal stresses caused by short circuit currents.

Table 5.21 Requirement of power for different capacities of STP (Activated sludge process)

STP capacity MLD	Power in kVA		Recommended Back-up supply (Essential loads, max 8 hours), kVA	Modular units		Approx. Daily Energy consumed kWh
	Lower value	Upper value		Number of Modules	MLD of each	
2.5	250		250	1	2.5	1,200
5	315		250	2	2.5	2,500
10	400	500	300	2	5	4,800
20	750	900	500	2	10	9,000
30	1250	1500	750	3	10	12,500
40	1500	1750	1000	2	20	16,000
50	2000	2500	1500	2	25	20,000
60	2250	2750	1800	3	20	24,500
75	3000	3500	2250	3	25	28,000

Note : The above power considers:

a) A peak load of 200% for 4 hours and 150% for 6 hours

b) A nominal lifting of 6 m at the beginning of process. If additional lifting is required suitable revision will be required.

Their cores are made with cold-rolled grain-oriented high-grade steel and copper windings. The resin is filled and cured.

The maintenance is easy due to non existence of transformer oil.

The standard accessories are lifting lugs, mounting channels, Ratings and Diagram Plate. HV and LV Bushings, Terminal Connectors and Temperature sensing devices etc.

The specifications are:

Frequency 50 Hz,

Primary Voltages: 33000/22000/11000 V;

Secondary Voltages: 230 / 430V;

Tappings: ±5% in steps of 2.5% or any % as per site requirement;

Impedance 2% to 5%;

Basic Impulse Levels: 60, 75, 95 or as per requirement.

5.12.2.9   Parallel Operation

More than one transformer can be used in a substation to share the power supply and to keep the transformer size within handling limits or to act as standby. This is termed as Parallel operation. It is usual to have two transformers of equal kVA capacity in parallel, where both can share the load, or one can be on load (on duty) and the second as standby (off duty). Even multiple transformers can be used, where one group share the load (on duty) at a time and the other group is standby (off duty). The off duty transformers are kept charged on the primary side and open on the secondary, to avoid the transformer becoming cold and to ensure that the unit is ready to take on duty at any time. Parallel operating transformers should have same voltage ratio, compatible vector group and sequence, same impedance, and usually same rating.

5.12.2.10   Instrument Transformer for Metering

These are used in substations to measure the primary current, voltages and other electrical parameters and monitoring these parameters gives an indication of the health of the system.

5.12.2.11   Instrument Transformers for Protection

These are used to measure accurately the load and system condition at any time and send control signals at any preset abnormal condition to annunciate or trip the main switchgear. The proper functioning of the protective system will save a lot of time, cost and interruptions by pre-empting a dangerous situation.

5.12.2.12   Substation Fencing and Lighting

Proper fencing with separate entry and exit gates shall be provided to prevent any unauthorised entry by persons or animals, etc. Usually chain link fencing is sufficient. The gates should always be kept under lock and the key should be in a prominent place.

5.12.2.13   Earthing and Lightning Protection

These are essential to the substation to safeguard the equipment and operating staff from any electrical shock. In addition, the earth fault sensing relays act based on the leakage current through the earth path which should have minimum resistance to current flow at such times. An underground grid of conducting mat is to be formed in the entire substation area, which is connected to several earth pits forming a low resistive path to any leakage current. Similarly, the top of the substation structures are to be connected by a wire grid to intercept lightning strokes and conduct it safely to the ground.

All the above equipments should be provided strictly as per regulations and guidelines under the Indian Electricity Act 2007, and appropriate power regulations.

5.12.3   Prime Movers - Motors and Motor Controllers

The pumps used in sewerage are driven either by electric motors or by diesel engines.

5.12.3.1   Motors

Electric motors are the most widely used and are mainly of three types, Induction (AC) motors, synchronous (AC) motors and DC motors. Amongst these, the Induction motors are the most common. Synchronous motors merit consideration when large HP, low speed motors are required. DC motors are used occasionally, especially for the speed variation duties.

5.12.3.2   Selection Criteria

The type of motor has to be selected considering various criteria such as the constructional features desired, environmental conditions, type of duty, simplicity and ruggedness of construction, endurance life cycle, capital and operating costs.

5.12.3.3   Constructional Features of Induction Motors

The most commonly used induction motors are of 'Squirrel cage' type. Normally, the starting torque requirement of centrifugal pumps is quite low for which this squirrel cage motors are suitable. Slip ring or wound-rotor motors are used where required starting torque is high as in positive displacement pumps or for centrifugal pumps handling thick sludge. The rotor is a circular cylindrical stack of laminated iron stampings with high magnetic permeability, shorted at both ends by a ring of aluminium or copper, and the stator is a cage of copper winding. The cage itself is again made of circular stampings with slots, which are aligned to allow copper winding to be inserted. The insulated windings are grouped into series and parallel sets based on the design and connected suitably to bring out the terminals. Normally, six terminals, two for each phase are brought out to the terminal box where the incoming cable can be connected, to form either STAR or DELTA connection. The rotor is positioned on bearings to rotate freely within the stator cage.

When the stator core is magnetized due to current flow in the winding a rotating magnetic field is created which pulls the rotor to rotate, following the rotating field. When power is disconnected, the stator core loses its magnetism, thereby the rotor stops slowly.

5.12.3.4   Method of Starting

Squirrel cage motors when started direct-on-line (with DOL starters) draw starting current about 6 times the full load (FL) current and in case of large motors, this could affect the system voltage and other running loads and may trip the system too.

If the starting current has to be within the regulatory limits specified by the power supply authorities, the squirrel cage motors shall be provided with a starter, which reduces the starting current.

5.12.3.5   Voltage Ratings

Table 5.22 (overleaf) gives the standard voltages and corresponding range of motor ratings.

For motors of ratings 225 kW and above, where HT voltages are 3.3 KV, 6.6 KV and 11 KV can be chosen, the choice shall be made by working out relative economics of investment and running costs, taking into consideration costs of transformer, motor, switchgear, cables and others.

Table 5.22 General motor ratings for various voltage ranges

Supply	Voltage	Range of Motor rating in kW	
		Minimum	Maximum
1 φ AC	230 V	0.3	2.5
3 φ AC	415 V	-	250
	3.3 KV	225	750
	6.6 KV	400	-
	11 KV	600	-
DC	230 V	-	150

Source: CPHEEO, 1993   N.B. When no minimum is given, very small motors are feasible.
When no maximum is given very large motors are feasible.

5.12.3.6 Types of Enclosure

The type of enclosure for the motor shall be as in Table 5.23.

Table 5.23 Protective enclosures and types of environment

Type	Enclo-sure type	Description of environment
Screen-protected drip proof	IP.11	Indoor, clean(dust free) environment
Totally enclosed, air cooled	IP.44	Protected against solid objects over 1 mm (tools, wires, and small wires), Protection against water sprayed from all directions - limited ingress permitted. Indoor, dust-prone areas
Totally Enclosed, Fan cooled,	IP.54	Protected against dust limited ingress (no harmful deposit), Protection against water sprayed from all directions - limited ingress permitted.
Outdoor application	IP.55	Protected against dust limited ingress (no harmful deposit), Protected against low pressure jets of water from all directions - limited ingress.
Submersible application	IP 68	Protected against ingress of any dust while exposed and sewage or sludge while submerged

Source:CPHEEO, 1993

Ingress Protection (IP) ratings are developed by the European Committee for Electro Technical Standardization (CENELEC)/(NEMA IEC 60529 Degrees of Protection Provided by Enclosures IP Code), specify the environmental protection as an IP rating with two numbers where the first digit is the protection from ingress of solid objects or materials and the second digit is the protection from ingress of liquids (water).

5.12.3.7   Class of Duty

All motors shall be suitable for continuous duty i.e. class S1 as specified in IS: 325 and additionally, it is recommended that motors should be suitable for maximum six equally spaced starts per hour and the motor shall be suitable for at least one hot restart.

5.12.3.8   Insulation

Class B insulation is generally satisfactory, since it permits temperature rise up to 80°C. At cool places having maximum ambient temperature of 30°C or less, motors with Class E insulation can also be considered. At hot places having maximum ambient temperature of above 40°C, motors with Class F insulation shall be considered. The present practice for HT Motors is to specify with class F (155°C) insulation with temperature limited to Class B (130°C).

5.12.3.9   Margin in Brake Kilowatts (BkW)

Motors are rated as per the output shaft horsepower (Brake kilowatts, BKW) but their rating shall be selected as to provide margins over the BKW required by the pump at its operating point as in Table 5.24.

Table 5.24 Margins for motor-ratings

Required BkW of Pump	Multiplying factor to decide motor-rating
Up to 1.5	1.5
1.5 to 3.7	1.4
3.7 to 7.5	1.3
7.5 to 15	1.2
15 to 75	1.15
Above 75	1.1

Source: CPHEEO, 1993

5.12.3.10 Electrical Switchgear and Control Equipment

The electrical equipment selected shall be adequate, reliable and safe, and the adequacy shall be determined by the continuous current required for the station-load and the available short-circuit characteristics of the power supply.

The reliability depends upon the capability of the electrical system to deliver power, when and where it is required, under normal as well as abnormal conditions.

Safety involves the protection to the plant personnel and the safeguarding of the equipment under all conditions of O&M.

These three aspects shall not be sacrificed for the sake of initial economy. The electrical system shall be designed with such flexibility as to permit one or more components to be taken out of service at any time without interrupting the continuous operation of the station. A proper selection of voltages in the electrical system is one of the most important decisions that will affect the overall system-characteristics and the plant-performance. The station bus bar voltage shall be at the level that is most suitable for the pump-motors, which constitute the major part of the load.

5.12.3.11   Switch Gear

The functions of a switchgear in a power distribution system include normal switching on, normal switching off, fault-tripping operations and equipment protection. Motor-starting function can sometimes be vested in the switchgear, but only when the frequency of starting and stopping is low or in applications where the motors are of such magnitude that no other equipment is suitable.

5.12.3.12   Switchboards

Various configurations of switchboards can be used in sewerage. Due to the distances between various components, a single switchboard is not feasible. Therefore, a master switchboard, with about 8 to 10 feeders will be located near the substation. The various feeders shall branch out to major load-centres in the plant where they will terminate in smaller sub-switchboards. This will help in suitable grading of fuses and protective relays so that the entire power supply is not interrupted due to a small fault in a remote section.

5.12.3.13   Starters

A starter for electric motor is a control device to start and stop a motor. The starting, protection during starting and running, stopping and any operational control during running of the motor are handled by the motor starter. Since the motor accelerates from zero speed to full speed in a short time, and sometimes drives a heavy load during starting itself, a proper starting method is required to protect the motor from electrical and mechanical stresses and possible failure. These can be of different types, viz., direct on line (DOL), Star-Delta, autotransformer and stator-rotor. Fully automatic starters are preferable. The stator rotor starter is meant for slip-ring motor and the others are used for squirrel-cage motors. A new generation of starters termed soft starters have been developed in the past decade with either reactance coil, or thyrister circuit to give a smooth 'ramp' of starting torque and limit the starting current to as low as 3 times the full load current. Such starters put less strain on the motor during starting and thereby extend the motor's life and reduce failure rate and are recommended.

The general guidelines regarding the use of starters for squirrel cage motors are given in Table 5.25 overleaf. Ingress Protection (IP) is applicable for starters too. Outdoor starters such as those for aerators and raw sewage screens shall be in weatherproof enclosure. Even flameproof enclosure (IP65) may be required where there could be methane gas exposure, like locations near digesters. All copper contacts shall be covered with heat shrinkable sleeves or petroleum jelly to protect them from the corrosive atmosphere prevalent in the STP to protect the copper parts from oxidation and resultant loose contact and damage to the connections.

Table 5.25 Guidelines for starters for squirrel-cage motors

Type of Starter	Percentage of Voltage Reduction	Maximum Starting Current	Ratio of starting torque to locked rotor torques %
DOL	Nil	6 × FLC	100
Star-delta	58%	2 × FLC	33
Auto-Transformer	tap 50%	1.68 × FLC	25
	tap 65%	2.7 × FLC	42
	tap 80%	4 × FLC	64
Soft Start	40% to 65%	3 × FLC	
Variable Frequency Drive	40% to 90%	2.5 × FLC to 5 × FLC	20 to 80

FLC= Full Load Current.

Note: As per torque speed characteristics of the motor, the torque of the motor at the chosen percentage of reduced voltage shall be adequate to accelerate the pump to the full speed.

The starting torque of a pump is usually low at about 20% to 25% of the full load torque. Therefore, even a voltage reduction of 60% to 65% during starting can accelerate the pump.

Source: CPHEEO, 1993

5.12.3.14   Capacitors

The electrical motors are inductive loads, which reduce the power factor. The induction motors work at a power factor of around 0.84 to 0.87 lag. Many power supply agreements stipulate a minimum power factor of load to avoid penalty, which is usually in the range of 0.9 lag. Therefore, for improvement of power factor to acceptable levels, appropriate capacitors shall be provided. Capacitors conforming to IS: 2834 are static units that can be located in the motor control panel or as a bank for a group of loads, or even at the substation level. Normally the capacitors are connected at the motor starter or switchgear so that it will come into the circuit simultaneously with the motor and go off the circuit when the motor is switched off.

In case of existence of backup power supply, the capacitor bank shall be connected to main Panel with automatic switching contactors to cut in or cut out capacitor units (APFC). Capacitors are provided to improve the overall power factor to around 0.97 lag. An improved power factor as near to unity as possible will reduce the reactive load, which is also otherwise metered as demand charges in a HT installation.

An overall power factor of unity can also be achieved by sensing HV side power factor and adding capacitors on the HV side. There shall be HT or LT capacitors based on the operating voltage of the motors.

5.12.3.15   Speed Control of Motors

The speed of an electric motor is dependent on the supply frequency and the number of magnetic poles created in the stator winding. It is given by the formula:

$$N = 120 \times f/P \tag{5.43}$$

Where

   f =   Supply frequency in cycles/second and

   P =   Number of magnetic poles created in the stator winding.

A normal 4-pole motor runs at a nominal speed of 1500 rpm. By selecting different pairs of poles from 2 to 8, nominal speeds between 300 rpm and 750 rpm can be obtained.

However, where the process demands a speed variation, variable speed drives are applied whereby the frequency is varied to change the motor speed. These are also called variable frequency drives (VFDs) and are made part of the motor starting circuit to act as a starting control and as a speed controller.

Other types of speed control like phase shifting type are also available. In the case of pumps in sewage duty, the change in plant load over a day may need different flow rate pumps for which variable speed motors may be considered. Variable speed pumps are efficient where the friction loss component is dominating as compared to the static lift. In most cases of STPs, the static heads are the dominant factor since the pipeline lengths are low and this reason combined with the sensitive electronic equipment associated with the power circuit and the corrosive environment make the application of variable speed motors for pump application as very few.

The variable flow can be more easily achieved by running one more pump or one less pump. VFDs are useful to control the speed of the motors controlling the surface aerators, air compressors or air blowers, as the case may be. This is required to save electrical energy to maintain only the required dissolved oxygen (DO) level at all times. An instrumentation controlled interactive automation is advisable in large installations.

5.12.3.16   Cables

The flow of power from transformer to switchgear and from there to starter and to motor and other related equipment like capacitors are through power cables and Table 5.26 (overleaf) gives guidance on these.

Consequent to progress in PVC and XLPE cable manufacturing technology, paper insulated, lead-sheathed, jelly-filled and other forms of cables are now discontinued in general and only PVC for LT and PVC/XLPE for HT cabling shall be used and are to be of aluminium conductor and armoured to protect from underground hazards. The size of the cable should be selected such that the total drop in the voltage when calculated as the product of current and resistance of the cable shall not exceed 3%.

The values of the resistance of the cable are available from the cable-manufacturers.

In selecting the size of the cable, the following points (overleaf) shall be considered.

Table 5.26 Types of cables for different voltages

No	Range of Voltage	Type of Cable to be used	IS Ref.
1.	1 phase - 230 V 3 phase - 415 V	PVC insulated, PVC Sheathed	IS: 1554
2.	Up to 6.6 KV	PVC insulated, PVC Sheathed	IS: 1554
4	11 KV 3 phase	XLPE- Cross Linked, Polyethylene insulated, PVC sheathed Cable	IS: 7098
5	1 phase – 230 V	Paper insulated, lead sheathed	IS: 692

Source: CPHEEO, 1993

a) The current carrying capacity shall be appropriate for the lowest voltage, the lowest power factor and the worst condition of installation i.e., duct-condition

b) The cable shall also be suitable for carrying the short circuit current for the duration of the fault. The duration of the fault should preferably be restricted to 0.1 seconds by proper relay setting.

c) Appropriate rating factors shall apply when laid in groups (bunched) and/or laid below ground.

d) For laying cables, IS: 1255 shall be followed.

5.12.3.17   Controls

The controls shall be simple, direct and reliable. Large pumping systems may have controls that automatically start and stop the pump-units and associated valves and auxiliaries. A proper hand-operated selector switch may also be provided to avoid over-working of any one pumping unit.

Liquid level controls generally employ floats, ceramic floats being preferred to metal floats as the latter are affected by the chemical action of the sewage.

All floats are subject to accumulation of grease and scum and shall be periodically revamped.

As a recent development, various other level sensors, such as, ultrasonic type, radar type, capacitance type, etc., are also being used.

The various functions, which a control-panel has to serve and the corresponding provisions to be made in the panel are detailed below:

1. For receiving the supply- Circuit breaker or switch and fuse units

2. For distribution- Bus bar, Switch fuses units, Circuit breakers

3. For controls - Starters, Level-sensors, Flow-sensors, Time-delay relays,

4.   As protections - Under voltage, over current, earth fault and Motor Protection Relays.

5. For indications and readings — Phase indicating lamps, Voltmeters, Ammeters, Frequency meter, Power factor meter, Temperature scanners. Indicators for state of relays, Indicators for levels, Indicators of valve positions, if valves are power actuated.

The scope and extent of provisions to be made on the panel would depend upon the size and importance of the pumping station. The space clearances shall be ensured as per Indian Electricity Act, rules and regulations.

5.12.3.18   Submersible Pump Motors

The submersible pumps are widely used and are monoblock pump sets with motor enclosed in a watertight compartment and the submergence in sewage serving the cooling purpose of the motor.

Since the motor casing is continuously cooled by the sewage, the motor can work at higher current loads without being overheated. The cable connection is also through a watertight gland.

The sewage submersible pump testing shall conform to IS: 9137 and IS: 1520.

The sewage submersible motor test shall conform to IS: 325 and IS: 4029.

Some salient features of submersible sewage pump are listed below:

a) The pumps are provided with maintenance-free anti friction bearings permanently grease-filled to sustain the axial and radial forces.

b) Impeller is of non-clog design capable of handling unscreened sewage and solids up to 75 mm size without clogging.

c) The mechanical seals have silicon carbide facing, and are effective in both directions of rotation.

d) The pump has a bolt-free connection facility for easy lifting and lowering. A flanged clamp with a vertical mating face is fixed to the beginning of delivery pipe at the low level of wet well. The pump delivery has a corresponding mating clamp with rubber ring on the vertical mating face. When the pump is lowered, the two mating faces get joined with the rubber ring to seal. Fixed clamp is given a slight taper as to make the joint water-tight as the pump is fully lowered and its own weight makes it tightly sealed.

e) All fasteners are of stainless steel.

f) The motor is a class F IP 68 protection enclosure with resin-impregnated windings.

g) Normally motors up to 65 kW are dry type with cooling fines and the cooling is effected by the surrounding fluid being pumped. For higher rated motors in-built closed loop liquid (water glycol coolant) cooling system are used. A separate impeller mounted on the pump shaft itself shall effect coolant flow within motor. An in-built heat exchanger without cooling fines, like the cooling jacket shall be provided for cooling of the coolant by the pumped liquid.

5.12.3.19   Single Line Diagram

A Single Line Diagram (SLD) depicts the power circuit from the supply point to the Switchgear, and up to the individual equipment. This also indicates the tappings for metering and protection, Circuit Breakers, Transformers, Generators, indications of where cables are connected, and important rating/specifications for the power equipment. Several electrical symbols are used in the SLD for notation of the various power equipment. All the Graphical symbols used for the various electrical components are standardized through 'IEC 60617- Graphical symbols for Diagrams' which contain some 1750 symbols. A typical single line diagram is shown in Figure 5-54 overleaf.

5.12.4   Backup Power Supply

All STPs shall be equipped with diesel generators. The rating shall be able to run continuously and give power supply to all the essential equipment of the plant (e.g., pumps, aerators, laboratory, etc.) plus lighting and the location shall be as near as possible to the major load centre or near the main LT panel to which the power will be supplied. The number of units shall be split into two or three generators instead of one large generator so that in case of requirement during lean load period, only one generator may be run to cater to the reduced load. Further, any one unit can be taken up for maintenance without the risk of total absence of standby facility.

Where the raw water pumping or terminal pumping station within the STP is considerably away from the other power-intensive units, two separate standby power supply systems can also be installed, one set for the pumping station and connected loads , and another for the rest of the plant. The generator shall have its own switchboard with an incomer and one or more feeder switches. The incomer shall receive supply from the generator through an incoming circuit breaker; the feeders will supply power to the recipient switchboard. This is required to protect the generator from any fault in the recipient load and protect the downstream load from any abnormal output from the generator.

5.12.4.1   Parallel Operation of Generators

When more than one generator is to be operated to share the load, the units have to be 'synchronized' before they are allowed to share the load. The synchronization means equal voltage, equal frequency and equal phase sequence. For this, a synchroscope shall be installed in the generator panel or the coupling panel, which will indicate the right time to close the circuit breaker.

5.12.4.2   Fuel Storage

Adequate fuel has to be stocked so that the bulk fuel storage normally caters to seven day's demand of fuel. Since the diesel is inflammable, adequate precautions shall be taken for storage, protection and upkeep of the storage facility.

5.12.4.3   Cost of Captive Power

Larger the generator size, lower is the cost per unit generated. However, larger units also consume more fuel during idle running. The cost of captive power generation is usually more than the cost of power obtained from electricity supply company.

LEGEND

11KV CIRCUIT BREAKER IN KIOSK/PANEL

DRAWOUT TYPE CIRCUIT BREAKER

ACB - FOUR POLE AIR CIRCUIT BREAKER

SWITCH /ISOLATOR — EARTHED

HRC LINK TYPE FUSE

THREE PHASE POWER TRANSFORMER

CURRENT TRANSFORMER

VOLTAGE TRANSFORMER

CABLE TERMINATION

SPRING RETURN TYPE CONTROL SWITCH WITH
RED, AMBER AND GREEN INDICATING LAMPS.

DIESEL GENERATOR SET

EARTH CONNECTION

INSTRUMENTATION

(A) AMMETER

(V) VOLTMETER   VS VOLTMETER SELECTOR SWITCH

(KW) KILOWATT METER   KWH KILOWATT HOUR METER

(COSθ) POWER FACTOR METER   (F) FREQUENCY METER

RELAYING   (R) 27 AC UNDER VOLTAGE RELAY

1. INCOMER BREAKERS 'A' AND 'B' AND BUS SECTION BREAKER 'C'
SHOULD BE PROVIDED WITH CASTLE KEY INTERLOCK TO ALLOW A
MAX OF TWO BREAKERS CLOSED AT A TIME.

2. GRID SYNCHRONIZATION WITH D G SUPPLY IS NOT ENVISAGED.
EITHER GRID SUPPLY ONLY OR D G SUPPLY ONLY WILL HAVE TO
ENERGISE THE MAIN PANEL. APPROPRIATE INTERLOCKS BETWEEN
THE INCOMER ACB TO BE PROVIDED.

3.a) UNDER NORMAL CONDITION, PANEL WILL HAVE INCOMER ACBs 'A'
AND 'B' CLOSED AND BUS COUPLER ACB "C" OPEN.

3.b) ON GRID SUPPLY FAILURE, ACBs 'A' AND 'B' WILL OPEN ON
UNDERVOLTAGE; D G SET WILL START AUTOMATICALLY. ACB 'D' WILL
CLOSE AFTER ENSURING THAT ACB 'A' AND 'B' ARE OPEN.
BUS COUPLER ACB 'C' WILL BE CLOSED MANUALLY. SIMILAR CYCLE
WHEN GRID POWER RETURNS MAY BE DETERMINED.
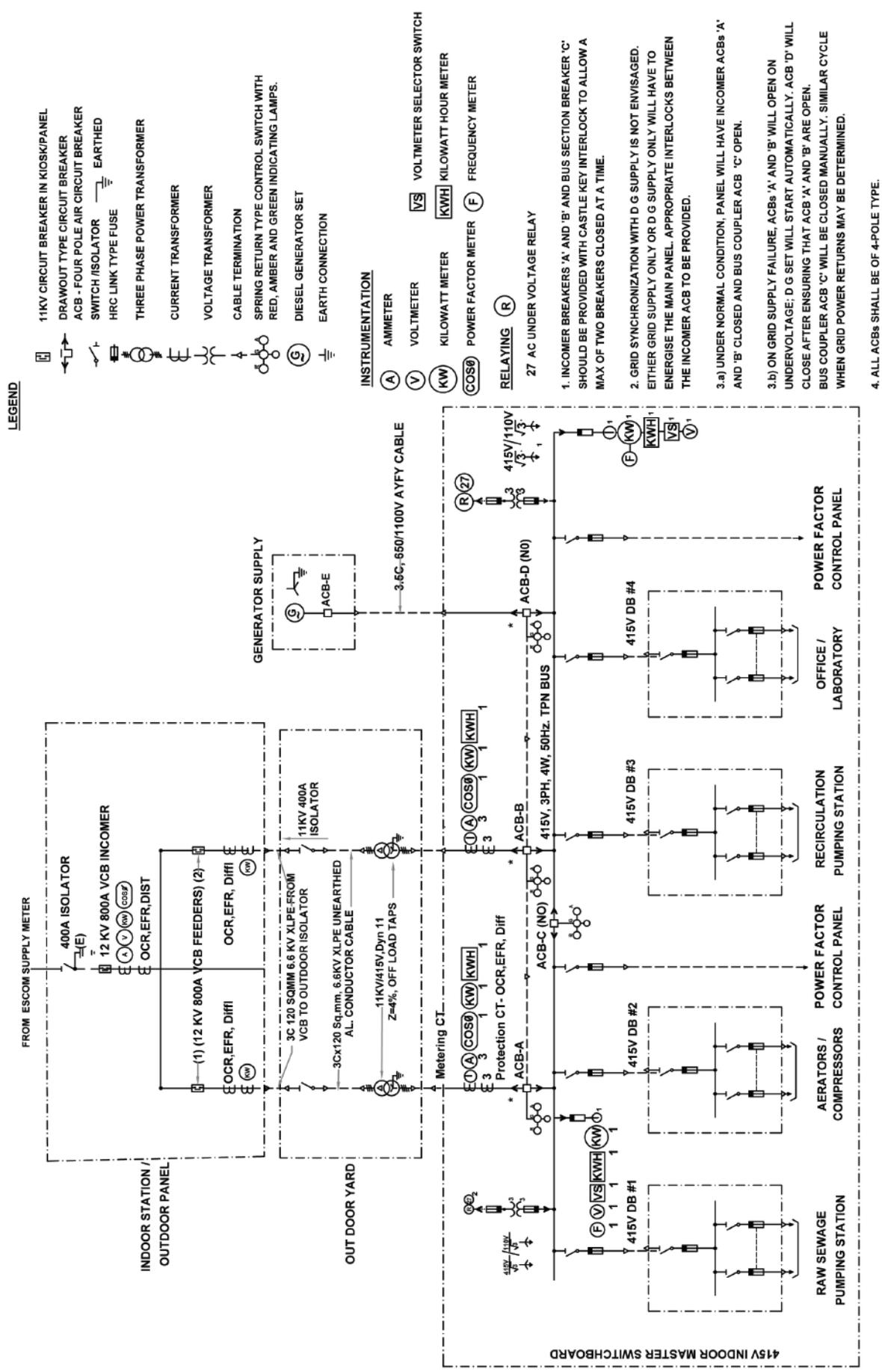
4. ALL ACBs SHALL BE OF 4-POLE TYPE.

Figure 5.54 Typical Single Line Diagram of 11 KV yard and master L.T. switchboard

On an average, one unit of energy will need between 0.25 to 0.4 litres of diesel, depending on its size. In addition, the generator also needs lubricating oil, maintenance, etc., and therefore, the use of generator shall be resorted to, only when necessary.

5.12.4.4   Electricity Room and Generator Room

The Electrical Switchgear room shall be well illuminated and well ventilated. The ceiling height is stipulated to be at least 1.8 m above the highest point of the switchgear to the lowest point of the ceiling. Generally, this is not less than 3.2 m, where use of tripod is not envisaged for moving/ erection of the panels. Even higher levels of ceiling are common to ensure better air ventilation, natural lighting and equipment handling needs. The clearance between side and rear of panels and the wall shall be not less than 700 mm in case of LT Panels and 1000 mm in case of HT panels.

Ample space is required to be available in front of panels for operation, monitoring and repairs. Where draw-out type breakers are employed, the draw-out distance has also to be considered. Entry of cable ducts to the rooms should be done in such a way as to prevent water entry through the duct. The duct can be of brickwork with internal plastering and angle supports to keep cables clear of the duct floor. Multi-tier cable racks are used to lay power and control cables.

Galvanized steel cable trays are also advisable as the ducts are mostly covered and can be humid due to the low ventilation. Earthing strips from switchgear should be taken through the cable ducts with clear identification. Battery rooms should be well ventilated with exhaust fan and have acid-resistant floor and wall tiles. The operator's table if located in the switchgear room shall be strategically located as to facilitate proper supervision.

5.12.4.5   Clearance from Statutory Authorities

For 11 KV and higher voltage substations, clearance to the layout drawings shall be obtained from The Jurisdictional Electrical Inspector of the Government before commencement of work. The written concurrence shall have to be obtained from the Electrical inspector again prior to commissioning, for compliance to the rules and safety aspects.

In case of generators, approval of the location and layout shall be obtained prior to erection and again before commissioning.

5.12.5   Automatic Mains Failure Panel

Failure of mains supply to the plant can occur at any time. In order to ensure that the treatment process is not unduly interrupted, an automatic changeover panel is installed. The automatic changeover panel is also called automatic mains failure panel (AMF) and it ensures that

a) When the main power supply is interrupted, the generator will start automatically after a certain time lapse to resume power supply to the plant.

b) When the main power supply is back, mains power supply will be resumed instantaneously after cutting-off the power from the generator. However, the engine should run for some more time on no load to facilitate the cooling down of the engine.

A set of changeover power contactors are normally employed for transfer of source between Mains and backup, ensuring that there is no faulty operation. AMF Panels are available from 15 KVA onwards. For successful operation of AMF Panel, the generator set and its controls have to be in good working condition, ready to start any time.

5.12.5.1 Sequence of Operation of AMF Panel

The sequence of operation of the AMF panel is described briefly below:

1. The mains supply source, which supplies power to the plant shall be constantly monitored by a mains voltage monitor. It will also monitor the readiness of the generator to start.

2. When mains voltage fails or drops below 70% to 80% of the rated voltage, the automatic control system shall give a starting signal to the diesel generator set.

3. The diesel engine will start. Once the diesel generator set reaches its operating speed and the alternator attains its operating voltage, a change over switching operation will occur through a set of relays whereby the mains supply switch will open and the backup supply switch will close. Thus, the load is transferred to the generator set. During this start-up and change over period, for a short time there will be no power to the plant during this operation.

4. Upon the return of the normal source voltage to the rated voltage or 90% of rated voltage for at least one minute (or any such stabilizing time), the changeover relays will activate opening of backup supply switch and closing of mains supply switch. In many cases this is done with such a short time gap that the plant runs uninterrupted.

5. The diesel engine continues to run on no load for some time. After a time-gap the engine stops. The automatic control system then resets itself and in readiness to start the engine in the case of the failure of the normal source.

6. In the event of failure of the diesel generator set to start/ deliver the power due to faulty starting within a specified period, there shall be an audio alarm. On hearing the alarm, the situation shall be investigated and remedial measures shall be taken without any delay. The changeover logic will be locked not to operate, until the generator problem is set right.

It is inevitable that all the electrical equipment will stop for a short while during the transition from the mains supply to the generator supply.

By the use of AMF Panel for automatic changeover of supply from grid supply to generator supply and vice versa, the interruption to operations is minimized. The station operator only needs to restart the components of the plant during the first changeover, and only monitor the smooth transfer during the second changeover.

A manual / automatic selection switch is also provided for manual operation in case of any problem.

The safety requirements of AMF panel are furnished in NEMA standard ICSL 2447.

5.12.5.2   Inbuilt Timers for Safety of the Operation

There are various inbuilt timers for the safety of the operation of the AMF. They are

• Start Delay or Blackout Timer: Time delay from mains failure to Diesel Genset starting

• Warm up Timer: Time delay from Genset running to load transfer on DG supply

• Mains Ok Timer: Time delay from mains sensing to load transfer on restoration of mains supply

• Cool down Timer: Time delay after load transfer on mains to DG shut down

• LOP Bypass Timer: Time delay during which LOP signal is NOT sensed

• Fault Relay Timer: Maximum time duration for sounding hooter after fault is sensed (If reset key is not pressed).

5.12.6   Uninterruptible Power Supply

The uninterruptible power supply (UPS) is an auxiliary piece of equipment, which provides back up power in electrical circuitry by drawing from a backup rechargeable battery or batteries for smaller drawals and through diesel driven generators for higher drawals. It is like an ambulance to take a patient to a hospital with oxygen and drips, etc., during the transit. Modern systems use a "double conversion" method of accepting AC input, rectifying to DC for passing through the rechargeable battery and then inverting back to AC for powering the protected equipment through a line-interactive UPS.

Most UPS below 1 kVA rating is line-interactive type where there is an additional multi-tap variable-voltage autotransformer which can boost or buck the powered coils of wire to regulate the output voltage fairly steady. However, the battery is charged only in high voltage mode but not in low voltage mode. For bigger systems, a synchronous motor/alternator can be brought in through a choke and energy stored in a flywheel. So that when the mains power trips, an eddy-current regulation maintains the power on the load as long as the energy of the flywheel can withstand.

The UPS can also be combined with a diesel generator to bring on the standby power after a brief delay and is referred to as DUPS and power during this delay to start the generator supplied by another UPS. Generally, the local electricity authority has provisions to supply power by a dedicated feeder instead of supplying the power from a public distribution grid. These feeders are given to hospitals, water treatment plants and fire fighting services etc., and effort shall be made to draw power for STPs from such dedicated feeders and have duplicate feeders to ensure that power supply to the STP is really uninterruptible.

However, for personal computers, SCADA systems, etc., it is necessary to provide an appropriate line interactive UPS.

In STPs, this type of power back up shall be ensured for at least the biological aeration systems and the return sludge pumps.

5.12.7 Instrumentation Facilities

The most important instrumentation needed in an STP is for the sensing of dissolved oxygen in biological aeration tanks to make sure that the microbes do not die off for want of oxygen. This is measured through a galvanic cell controlled probe principle as in Figure 5-55 and a hand held meter with a probe as in Figure 5.56 and the probe can also be used in the laboratory in a BOD bottle.
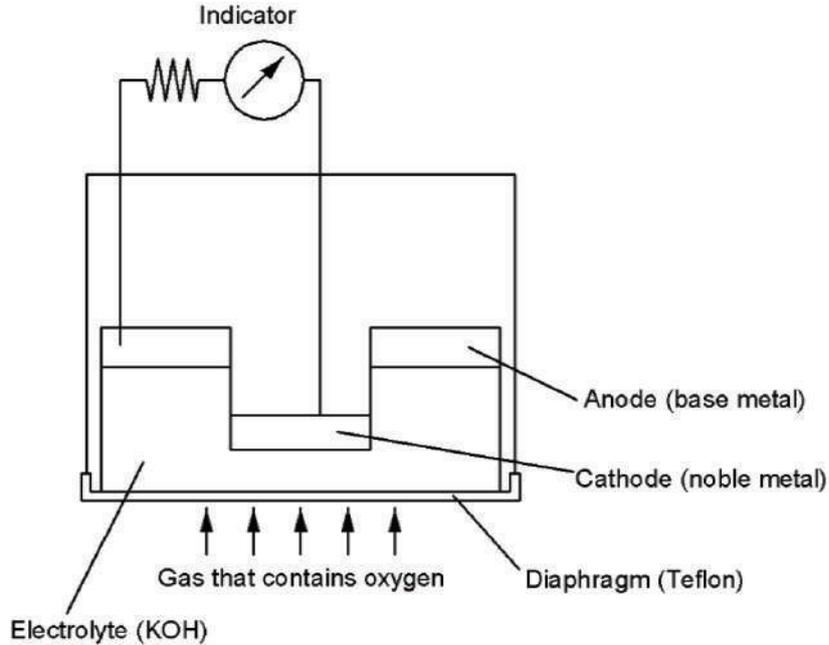
Source: Yokogawa Electric homepage

Figure 5.55 Principle of a dissolved oxygen probe

Source: M/s YSI catalogue

Figure 5.56 Hand held DO meter with probe for field use as well as laboratory use in BOD bottle

5.12.7.1 Principle of Working of DO Probe

When the probe is dipped into the mixed liquor of the aeration tank or into the BOD bottle containing the sample, the dissolved oxygen permeates through an oxygen permeable membrane covering the tip of the probe.

Then it enters the electrolytic solution in which an anode (base metal) and cathode (noble metal) are adjacent to each other, a current proportional to the quantity of DO is generated and is measured by the electrical circuitry and pre-calibrated to display the DO concentration directly.

5.12.7.2   Advantages and Disadvantages

The advantages are the detecting system is compact and portable with the cable length between the meter and the probe being available even up to 30 m. The disadvantage is the probe has to be cleaned almost every week and fresh membrane disc replaced with new electrolyte solution and the tip of the electrodes gently scraped to remove adhesions and oxidative residues.

5.12.7.3   Standard Procedures

The meter can go on, but the probe needs frequent attention. In the field, the D O can be measured at any depth and at any co-ordinates of the aeration tank to get an idea of the uniformity of D O and in case there are very low values in a zone, it may indicate that the air diffusers might have got choked. In order to do so, the probes can be tied securely to light weight but rigid pipes and immersed at the chosen location by standing on the platform. However, in the case of aeration tanks using surface aerators, these should be used only near the sidewalls and floor near the walls as otherwise the cable may get entwined in the swirl of the mixed liquor and may even draw the operator into the tank.

5.12.7.4   Minimum Velocity

There has to be a velocity of the liquid across the probe surface to induce the required hydraulic shear. In aeration tanks this is automatically obtained. In BOD bottles, a magnetic stirring glass capsule iron needle is first dropped into the BOD bottle and the bottle mounted on a magnetic stirrer so that the liquid inside the bottle is stirred to induce the required velocity. Alternatively self stirring probes are also available which have a rotating fine brush eccentrically to the probe axis which serves to agitate. The illustrations are shown in Figure 5.57 and Figure 5.58 overleaf.

Source M/S YSI catalogue

Figure 5-.57 Setup with magnetic mixer and mixed needle for agitation

Source M/S YSI catalogue

Figure 5.58 Eccentric axis self stirring probe with the D O probe to generate velocity in the liquid

5.12.7.5   Slime Build up on Probes in Aeration Tanks

The probe if left into the aeration tank will have a tendency to build the slime onto the probe membrane and hence the probe has to be taken out and scrubbed gently and put back every day. This can be got over by using the self-stirring probes as in Figure 5.58

5.12.7.6   Instrumentation

Concerning instrumentation, the D O probe can be fixed at a desired location inside the aeration tank and the output signal of 4 to 20 mA can be relayed to the meter in the operator room and in turn can be hooked onto a desktop computer. The DO can be either measured and displayed 24 × 7 or checked at random by the operator.

5.12.8   Instrumentation Facilities – BOD

There are sophisticated BOD measuring analyzers, which use the principle of measuring the rate of initial D O depletion for about 10 to 15 minutes inside the BOD bottle. This is done by the D O probes fixed to the bottle instead of the usual ground glass corks and kept inside the incubator and thereafter extrapolating the same to the desired conventional 5 days or the recent 3 days while the incubator maintains the required temperature of 20ºC or 27ºC respectively. The extrapolation software is microprocessor based and is pre-set for a given sewage by initial calibration and recalibrated as often as needed by correlating with actual BOD values measured after the 5 days and 3 days.

These instruments help in getting a quick idea, instead of waiting for at least 3 days, if there are problems at the field STP like dropping of DO, poor settling due to sludge bulking requiring immediate adjustments of ratio of sewage to return sludge, changing excess sludge bleed, etc. These are stated to be versatile enough for direct field use. The only issue is when procured at a high cost, the repairs are to be borne by the O&M only and if these are part of 5 year procurement with replacements included, it may be worth. The portable BOD instrument is shown in Figure 5-59 overleaf.

5.12.9   Instrumentation Facilities – pH

In STPs, it will be useful to install a remote monitoring system for pH and residual D O in the biological aeration reactor subject to the day-to-day preventive maintenance against slime build up as in sub section 5.12.7.5.

Source: M/s HANNA

Figure 5-59 Portable BOD test instrument

5.12.10   Non-instrumentation Method of Quick BOD Measurement

The COD test can be completed in two hours with a one-hour reflux and a graph can be constructed for a given sewage showing the COD Vs. BOD at various stages of treatment and a COD reading obtained after two hours can be used to correlate the likely BOD values for a quick field check.

5.12.11   Flow Measurement

5.12.11.1   Magnetic Flow Meters

Magnetic flow meters work on the principle of electromagnetic induction. The induced voltage generated by an electrical conductor in a magnetic field is directly proportional to the conductor's velocity. Thus, the sewage is the conductor and these meters are suitable at almost all piping like, raw sewage, settled Sewage, primary sludge, return activated sludge, waste activated sludge and treated sewage. These are non-invasive and used in almost all pipelines, but of course, initial calibration is needed. The output is the standard 4 to 20 mA signal, which is relayed to the central monitoring system.

5.12.11.2   Ultrasonic Flow Meters

When ultrasonic impulses are released onto a pipe surface carrying sewage, the impulses are deflected along the flow direction based on the velocity of the flow before they impinge on the opposite sidewall of the pipe. The time taken is measured and is correlated to the velocity and then to the diameter of the pipeline and hence the flow rate is arrived at. Like magnetic flow meters, these are also non-invasive and used in almost all pipelines but of course, initial calibration will be needed. The output is the standard 4 to 20 mA signal, which is relayed to the central monitoring system.

5.12.11.3   Ultrasonic Level Sensors

These also work on the same principle as above and the time taken to reach the water surface and get back to the sonic emission probe mounted on top of a channel is used to measure the depth of the liquid surface. By integrating with the depth of the floor of the channel from the probe, the depth of sewage flow is arrived at. These are useful in Parshall flumes in raw sewage channels. The output is the standard 4 to 20 mA signal, which is relayed to the central monitoring system. This signal from ultrasonic level sensor can be fed to a microprocessor, which can be programmed to give output as flow in an open channel like the Parshall flume.

5.12.12   Supervisory Control and Data Acquisition System (SCADA)

SCADA is an acronym for Supervisory Control and Data Acquisition. This presents the data as a viewable and controllable system on the screen of a computer. The data thus collected can be stored and analyzed for optimization of the process and for better real time process control. This assists plant-operating personnel by monitoring and announcing off normal conditions and failures of equipment. This allows the operators to perform calculations based on the sensor inputs. Using the stored data daily, weekly and monthly reports can be prepared. It also allows the operator to know the state of a process and an alarm associated with it. It is also possible for an authorised operator to intervene and operate equipment at a remote location through a remote terminal unit (RTU) under this SCADA network.

A typical SCADA communication overview is shown in Figure 5.-60.

Source: Kruger

Figure 5.60 Typical SCADA communication overview

5.12.12.1 Components of SCADA System

The various components of a SCADA system are

a) Personal computers: These are used by the operator to view the data acquired and allow the operator to control and improve optimization.

b) Programmable Logic Controllers: They control the outputs based on the inputs being monitored in the required sequential steps and it also communicates with the personal computers.

c) Modems: They are used to transfer data from the sewage-treatment plant site, to the centralized control station.

d) Remote terminal units (RTU): They allow the central SCADA to communicate with the various instruments at the sewage treatment plant. It controls, acquires and transfers data from the process equipment at the site, in conjunction with the central SCADA.

5.12.12.2   SCADA Software

There is standard SCADA software available which can be installed in application servers at the plant site, but they should be capable of controlling and monitoring the various instruments. The data acquired from the RTU should be displayed in the SCADA screen and the logs of each site station measurement should be transferable using data export to data base processing software like Oracle, Microsoft Excel, etc. It should also support internet connectivity for data transfer.

5.12.12.3   SCADA Security Level

In order to prevent misuse and to restrict access to a site station measurement there should be privileges to the various users of the SCADA. Typically, there are three levels, which are (1) Operator Level, (2) Engineer Level and (3) Manager Level.

5.12.12.3.1   RTU Security Level

The RTU communication port should have a configurable access level for its security. The minimum access levels are described in the following sub-section. These access levels are required to control read and write access to that port. Hence, once all the RTU ports are configured with these varying access levels depending on the requirement, then it becomes secure against unauthorised usage. It can be re-configured only after unlocking the RTU common port.

5.12.12.3.2   Minimum Access Levels of Programming and Configuration Interface

The interface shall have the minimum varying levels of user access that can be configured by the system controller. They are:

a) Unlimited Access: This will allow the user to read and write all RTU configuration parameters such as local, network and system registers, Hardware input and output registers, event logs and logic programs.

b) Access without configuration: This will allow reading and writing of all RTU configuration parameters except system registers and ladder logic.

c) Access limited to only reading the RTU parameters

d) Access limited to RTU port configuration.

5.12.12.4   SCADA User Interface

The SCADA system shall permit the user to access displays via printing device and/or soft key menus with a choice of function keys, cursor, control keys or any key on the keyboard. The system shall support operator access to multiple displays at one time, including split screens where the operator may view more than one process area at a time and permit pop-up displays.

The operator shall be able to have access to context sensitive help at any time during operation of the system. The operator shall be able to access multiple data sources/items with a single tag name.

5.12.12.4.1   Command/Control Functions

The system shall allow the user to control a specific set point or to adjust a set of points depending on the operating limits. Control of individual set points shall be enabled based upon a user's security level.

5.12.12.4.2   Display Capability

The system shall allow the user to view animated graphics for process templates including valves, meters, etc. The system shall support the capability for the operator to view scanned images and be possible to animate these images.

5.12.12.4.3   Text Description

The system shall support use of true-type scale able fonts that may be scaled according to the desired size of the text. The fonts shall be loaded by the operating system.

Text shall be able to blink based upon any user definable condition occurring in the system such as an alarm on a particular set point.

5.12.12.5   Alarm Capabilities

5.12.12.5.1   Alarm Display Capability

The system shall support alarm display capability on the display. Current alarms shall be available as an alarm summary object and a chronological summary of alarms shall be available.

It shall be possible to inform the operator of an alarm condition via an audible tone, a pop-up display or any combination of animation types on the screen.

Alarm acknowledgement may be performed on all alarms, alarms in a single group, and alarms in a collection of groups in defined in alarm group hierarchy or on a point-by-point basis.

5.12.12.5.2   Alarm File Capability

Alarms shall be logged to a file for future viewing or review of alarm history data. The user shall have the capability to review the file for cause and event analysis. The alarms that are logged shall be configurable from a choice of the parameters listed during configuration.

5.12.12.5.3   Alarm Printing Capability

Alarms shall be allowed to be printed and the format shall be configurable and made up of any of the parameters listed during configuration.

5.13   DISTRIBUTED CONTROL SYSTEM

5.13.1   Description

For a fairly large sewerage system with many pumping stations, STPs, etc., it is possible to have a centralized control station to monitor, data logging, interfere and control the various operations by a Distributed Control System (DCS), with a network of PLCs and RTUs.

The DCS developed in 1975 was first used by pulp and paper mills and is rather widely used off late. It consists of distributed microprocessor based single loop controllers connected to a shared video-based operator station. It has substituted analogue controllers with Direct Digital Control (DDC). The video based operator station eliminated the need of large instrument panels, which was important prerequisite for operators.

DCS is the best choice for a system with many operational units (say, sewage pumping stations) at various locations away from the main control station (say, at the STP), linked through various methods of digital communication. However, this is complex and expensive, but provides various controls, functions and other reliability features.

Presently, DCSs are available in miniaturized versions for multitasking, multivariable, multi-loop controller used for process control. It is a functionally and geographically distributed system. Equipment making up a DCS is separated by function and is installed in multiple working areas.

The operator can view information transmitted from various units and displayed on a video display unit (VDU) and can change control conditions from a computer key board.

5.14   PROGRAMMABLE LOGIC CONTROL

5.14.1   Description

For a single operating unit (say, sewage pumping station), the Programmable Logic Controller (PLC) is the very vastly employed automatic controller, helpful in eliminating continuous human monitoring and control.

Various matrix of operations are pre-programmed with various inputs from instruments, such as level transducer, pressure transducer, flow transducer, motor bearing temperature monitor, vibration monitor, etc.

PLC is to change, than re-lay panels. This would reduce the installation and operational cost of the control system compared with the electro-mechanical relay system. PLC offers the advantages such as, ease of programming and re-programming, programming language is based on re-lay wiring symbols familiar to most operating personnel, high reliability and minimal maintenance, small physical size, ability to communicate with other computer systems, moderate low initial investment cost and availability of modular designs.

5.15   CORROSION PROTECTION AND CONTROL

5.15.1   General

Corrosion is the phenomenon of the interaction of a material with the environment (water, soil or air) resulting in its deterioration. There are many types of corrosion, the major types being galvanic, concentration cell, stray current, stress and bacterial.

Sewage collection and treatment systems are more prone to corrosion in view of the nature of the sewage. Raw sewage contains solids like grit that cause abrasion in sewers, pumps and their components, thus removing the protective coatings. This exposes the metal and accelerates the corrosion process. Hence, corrosion control becomes important in SST. It is particularly severe in areas where sewage strength is high, sulphate content of water is substantial and average temperature is above 20ºC. The corrosion problem in SST can be categorized as (1) Corrosion of material of sewers and (2) Corrosion of metal and concrete in treatment plants. The corrosion of sewers and relevant prevention and control are described in detail in section 3.69 in this manual. Corrosion in case of treatment systems is described below.

5.15.2   Corrosion of Treatment Systems

The important units from the corrosion point of view are civil tanks for raw sewage collection, clarifiers and sludge digesters. In addition, all metal parts like in the pumps, valves, screens and grit chamber equipment are also important.

5.15.2.1   Raw Sewage Collection and pH Neutralization

In general, raw sewage pH will always be near neutral in pH and is not corrosive. Large variations in acidic or alkaline pH can result only when huge industrial wastewaters get into sewers. The manual clearly recommends in section 3.8 that allowing industrial effluents into sewers shall be discouraged. If such a contingency is foreseen, it may be only for a brief period before the entry is detected and cut off. In such cases, sodium hydroxide shall be used if the pH is corrosive and hydrochloric acid shall be used if the pH is alkaline. The neutralizing chemicals would need to be stored in acid or alkali resistant containers and the solutions led to the neutralizing tank by non-corrosive, thick walled PVC piping.

5.15.2.2   Clarifiers

The floor bottom of the clarifiers is scraped by mechanical scrapers in order to divert the sludge to the central sludge pit. These scraper arms and the squeegees are constantly immersed in sewage and are not subjected to severe corrosion because they are not exposed to the air.

The specification for the steel used for the underwater mechanisms should be carefully drawn to ensure maximum protection from corrosion. It is normally specified that all the steel below liquid level shall be at least 6 mm thick. It is a good practice to keep all chains, bearings or brackets above the liquid surface. All castings in the drive mechanism should be of high-grade cast iron. It is also possible to give cathodic protection to the scraper mechanism either by sacrificial anode or by impressed current. The choice of either of the method will depend upon the comparative costs. In any case, the cost of such a protective measure will not be higher than the cost of good quality acid resistant paint.

5.15.2.3   Sludge Digestion

Corrosion problem in sludge digestion tank is described in section 6.11.1 in this manual.

5.15.2.4   Activated Sludge

In the activated sludge plant, oxygen is provided to the sewage either by surface aeration or by compressed air system. In the compressed aeration system, air supply is normally provided through mild steel pipelines. Though the air is filtered and moisture is removed before sending to compressors, still there can be problems and these can be minimized by use of air supply pipelines of non-corrosive material.

In surface aeration, proper material selection and coating are necessary for protection of the exposed parts of the aerator blades. It may be mentioned here that the protective coating has to be applied at regular intervals, since it is found that such coatings have very short life. PVC lining may not be easy to provide due to the shape of rotor while fibre glass lining can be relatively easier. For floating aerators, it is desirable to have corrosion resistant lining, such as fibre glass for the floats.

5.15.2.5   Attached / Fluidized / Immobilized Media Systems

In these systems, the mechanical components include the header, the distribution arm and the distribution nozzles. The header and the distribution arm are normally of mild steel and should be protected from corrosion by proper painting.

The corrosion and the resulting blockage of distribution nozzles are of common occurrence. This can be avoided by selection of corrosion resistant materials such as brass or PVC for nozzles.

5.15.3   Sewage and Sewage Pumps

For pumps and pumping equipment, proper materials selection is of paramount importance. The pump casing is normally of close-grained cast iron capable of resisting erosion of abrasive material in the waste. For handling sewage and other corrosive wastes, the impeller is generally made of high-grade phosphor bronze or equivalent materials. The wearing rings for impeller should be of good corrosion resistant materials such as bronze. The shafts are normally made of high tensile steel and replaceable shaft sleeves are recommended.

For pump and pumping equipment, painting is the usual protective measure. Both, the interior and exterior surfaces of pumps should be painted after rust, scale and deposits are removed by sand blasting, wire brushing or rubbing with sand paper

5.15.4 Preventive Maintenance

It will be seen from the above that anti-corrosive paints, coating and linings have to be used in various equipment to prevent corrosion. The paints, coatings and linings require periodical renewal, Proper maintenance demands that a schedule be drawn up so that the operator may abide by it and undertake repainting or cleaning at appropriate intervals without waiting for corrosion to become obvious.

5.15.5 Piping Requirements in Treatment Plants

Piping requirement in sewage treatment plants range from sewage and sludge conduits, drains and water lines to chemical process piping, if any. Materials for various pipeline applications are given in Table 5.27 (overleaf). In order to facilitate identification of piping, particularly in the large plants, it is suggested that the different lines shall be colour-coded. The contents and direction of flow shall be stencilled on the piping in a contrasting colour. Typical colour code appears below (WEF, MOP8, 2010):

- Orange: energized equipment and flammable gas lines

- Blue: potable water

- Yellow: chlorine

- Black: raw sludge

- Brown: treated bio solids

- Purple: radiation hazards

- Green: compressed air

- Jade green: non-potable process or flushing water

- Grey: sewage

- Orange with blue letters: steam

- White: traffic and housekeeping operations

- Red: fire protection equipment and digester gas lines

5.15.6 Modification of Materials

Normally, choice of materials shall be suitable under the circumstances likely to be encountered and commensurate with economy. If justified economically, corrosion resistant construction material can be used initially, as this may not require any additional protective coating frequently.

Stainless steel, aluminium and plastics are examples of materials of this nature. It is possible that the use of such corrosion-resistant materials would be cost-effective in the long run.

However, in treatment plants, it is found that it is usually less expensive to use ordinary structural steel to which protective coatings are applied.

Table 5.27 Piping Materials in Sewerage and Sewage Treatment

Typical Application	Concentration in %	Materials
Influent	0.5 to 2	C, CL, RCP, RC, VC
Secondary Solids	0.5 to 2	C, HP, PE, CI
Primary solids	0.2 to1	C, G, T, HP, PE, D, CI
Thickened Sludge	4 to 10	C, HP, PE, T, CI, D
Digested Sludge	3 to 10	C, HP, PE, T, CI
Chemically treated sludge	8 to 26	C, HP, PE, H, CI
Dewatered sludge	8 to 25	C, CI
Heat Exchanger	< 0.1	S
Spray irrigation	< 0.1	C, CI, T, A, HP, PE
Chemical Process Piping		C, CI, S, G, T, HP, PE, H, D
Aluminium Sulphate	15 to 22	C, D, H, HP, PE, T, S
Calcium Hydroxide	63 to 73	C, CI, D, G, H, HP, PE, S, T
Calcium Hydroxide	85 to 99	C, CI, D, G, H, HP, PE, S, T
Sulphuric Acid	93	S, G
Ferric Chloride	59 to 98	C, H
Sodium Hydroxide	73	C, S, H
Carbon Slurry	20 to 30	G

C - Carbon Steel   S - Stainless Steel
G - Glass Lined   A - Aluminium
T - Teflon lined   P - High Density Polyethylene
PE – Polyethylene
CI - Cast Iron   D - Ductile iron
H - Plastic or rubber hose   RCP- Reinforced Plastic mortar
RC - Reinforced concrete   VC - Vitrified clay

Source: CPHEEO, 1993

5.16   REHABILITATION OF SEWAGE TREATMENT FACILITIES

5.16.1   Criteria for Rehabilitation of STPs

In simple terms, the word rehabilitation refers to a situation that needs cure. Thus, rehabilitation shall be specific to non-performance and not a wholesale rehabilitation. Rehabilitation may not include capacity upgradation, but it can be other way about. In any case, except for the electrical control panels and cabling, all other rehabilitation efforts in STP should be taken up inter-alia between all unit operations, process technology and mechanical equipment all at the same time, instead of piece meal. Instances like the burn out of a motor winding are exceptions to be attended immediately and will come under the terminology of repairs and not rehabilitation. To this end, the primary responsibility of deciding whether it is rehabilitation or repair or upgradation is to be owned up at designated levels of authority before proposals are initiated.

As the overall administration of the STP is a complex discipline involving academicians, technocrats and financial managers, it is necessary to bring about the programme under a joint responsibility as in Figure 5.61 overleaf.

5.16.1.1 Upgradation of Facilities

Upgradation of STPs would normally arise in regard to capacity upgradation and there are technological strategies in hand in the present context. Some possible options are:

1. A conventional ASP can be upgraded without increasing the footprint by opting for an MBBR to be inscribed in the aeration tank and duplicating the hydraulic piping and pump sets and the primary clarifiers can be modified as rim flow clarifiers and secondary clarifiers inscribed with tube settlers.

2. The effluent quality of the existing ASP can be upgraded by adding tertiary coagulation and/or sand filtration, ultrafiltration.

3. Existing UASB can be upgraded by dismantling final polishing units and replacing it with activated sludge process or its variants such as SBR (A A Khan et al., 2012), SAF, MBBR, etc.

4. Stabilization ponds can be upgraded to aerated lagoons on extended aeration mode with bund partitions, to carve out the sedimentation zones.

5. Aerated lagoons can be upgraded with bio-towers inscribed at one end and boosting the delivery head of pump sets or replacements.

6. Large diameter old trickling filters can be upgraded into conventional ASP by constructing a radial wall and allowing the mixed liquor to flow annularly and equipped with floating surface aerators.

7. Conventional aeration tanks can be partitioned to change the process to contact stabilization and maximize treatment capacity for the same volume. The partitioning has to be by inscribing a separate structure for contact aeration preferably circular and without opening out the reinforcements of the old tank base slab or sidewalls.

- Understand the Importance of and Develop Specific Study Plan for Ascertaining Treatment Efficiency for consent to operate parameters
- Identifying shortcomings for complying with consent to operate conditions
- Ascertaining the adequacy of pump sets Vs process flows at all locations
- Ascertaining the oxygen transfer deficiencies of the aeration system
- Ascertaining the shortfalls in check measurements of laboratory results
- Ascertain safety infrastructure and first aid kit deficiencies
- Verify the responses from higher ups to requests from field staff

↓

| Assemble the STP Team keeping it cute in numbers and with persons known to take initiative | → ← | Review the causative reports, and arrive at prima facie justifications |

↓

| Fact finding mission of the team with all members being present simultaneously at site | → ← | Interactions within the team to decide on additional scope of insights at field & going through it |

↓

- Team develops the justifications for required rehabilitations of civil, mechanical and electrical items
- Induction of newer items of works and equipment
- Redundancies in existing system which are draining the budget to waste
- Deletions of items of work, personnel and electrical items with no need
- Priority listing of procurements, works, personnel staffing
- Formulate an Immediate works programme
- Formulate a time bound master programme

| Competent authority review, debate and accord | → | Competent authority sanctions the budget and designates officers |

| Reports justifying the variations if needed and recommending the variants to be sanctioned | | Independent team keeps tag of compliance of rehabilitation work and lists variations needed |

↓   ↓

Joint completion report on aims, objectives, compliances, deviations, deletions & recommendations for future to avoid such contingencies arising again
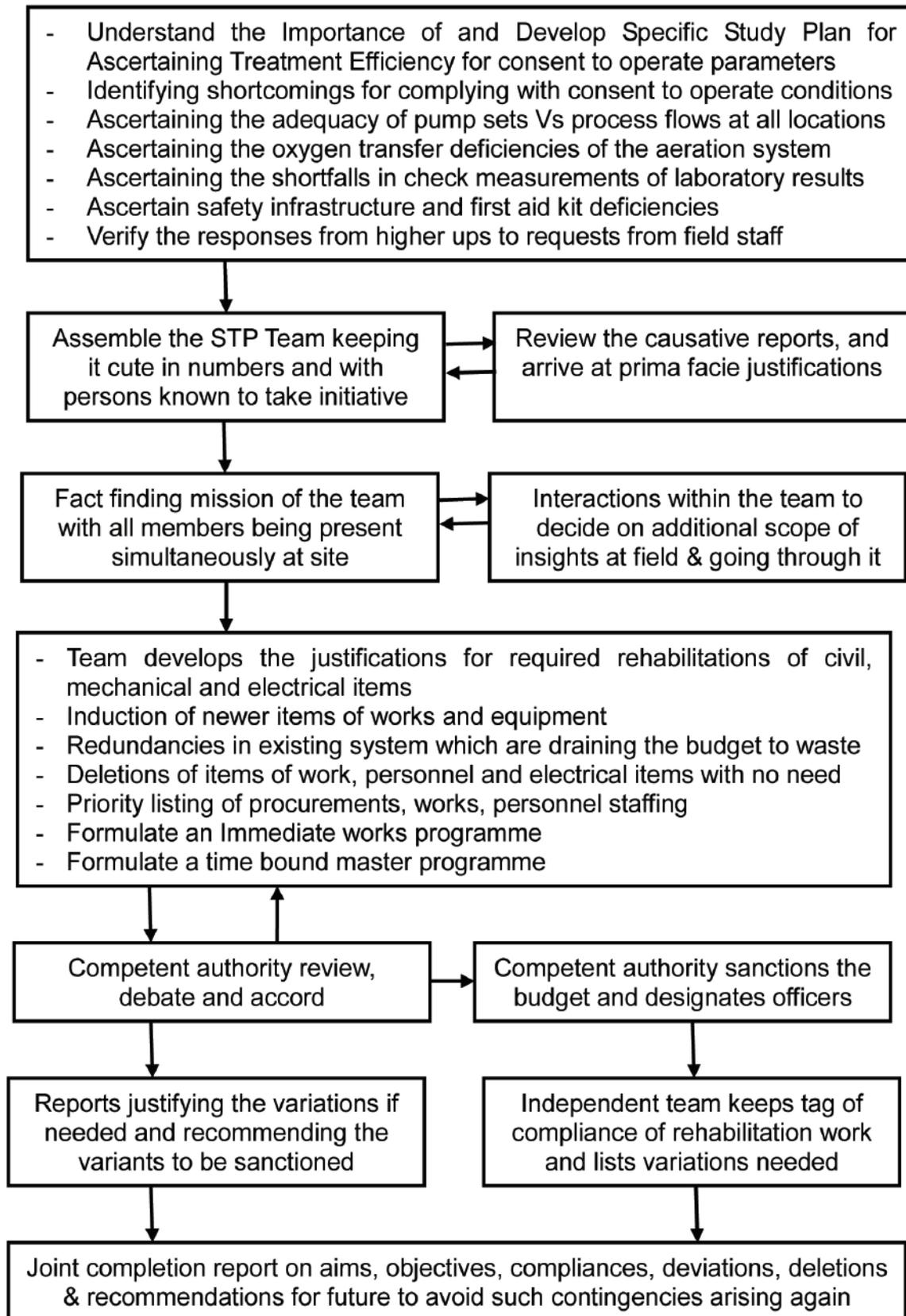
Figure 5.61 Algorithm for rehabilitation programme of STPs

8. The Melbourne Eastern STP upgrade is often cited in literature. It is from a conventional primary-aeration-secondary to online add on tertiary by ozonation-biological filters-ozonation-UV-chlorination before selling the reclaimed water to retailers who in turn sell it to end users. The biological filters used here are an unique component to biodegrade the residual organic matter and reduce ammonia, oil and grease, foam, litter and solids. The final water is to be used in farms, market gardens, vineyards, golf courses and sporting grounds in the city and at the plant, for every day operations to clean screens, to wash down work areas, for cooling and to water the grounds. The excess treated sewage will be discharged to the ocean where water contact sports will continue to take place. The significance of this to the Indian situation lies in the residual levels of phosphorous and nitrogen because of the higher concentrations in sewage due to lower per capita water supply and hence, mere add on units like the above may not be adequate and specific nutrient removal enhancements will also be needed.

9. Each situation needs to be approached individually and the question of following another location's experience has to be tempered with reasoning.

10. The other issue is the procurement of the recent patented technologies, which of course have to be secured against reasonable competition anyway.

5.16.1.2 Energy Saving Measures

In the case of new STPs, it is preferable to design the treatment process to include primary treatment and secondary treatment with an F/M ratio in the conventional regime whereby the blended sludge becomes available for biomethanation in digesters and generation of electricity there from. This saves the electrical power needed for the STP. The 40 MLD ASP at Chennai is reported as performing on these lines. The economics of such gas generation and utilization is influenced by the capacity of the plant and the raw sewage BOD and VSS.

Hence, in the preparation of DPRs, proposals for such utilization of electrical generation should be carefully weighed by a net present value (NPV) of the costs of (a) Hydrogen Sulphide stripping, Gas/dual fuel engines, repairs and renewals of these additions, their inherent O&M and electrical power needs, establishment costs and (b) cost of electricity that will otherwise be payable to the local electricity authority over the design period cited in Table 2.1.

5.17 CARBON CREDIT

This term qualifies the holder to emit one ton of carbon dioxide into the atmosphere and is awarded to institutions or countries that have reduced their greenhouse gases below their emission quota, which literally means emission standards. These carbon credits can be traded in the international market at their current market price. There are firms that have earned carbon credits and offer them to other firms who are interested in lowering their carbon emissions on a voluntary basis. They purchase the credits from an investment fund or a carbon development company that has aggregated the credits from individual projects. Buyers and sellers can also use an exchange platform to trade, such as the Carbon Trade Exchange, which is like a stock exchange for carbon credits. The global and Indian position are presented in Figure 5.62 and Figure 5.63 overleaf.

Carbon Credits Market : Registered Project Activities under CDM by Scope(World),
2008-09

Others include projects related to
Afforestation and reforestation, Metal
Production, Transport, Construction,
Energy Distribution etc.

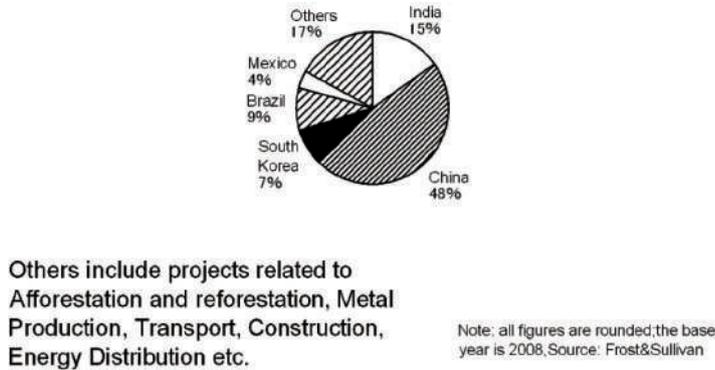Note: all figures are rounded;the base
year is 2008,Source: Frost&Sullivan

Figure 5.62 Country wise carbon credit market as of year 2009

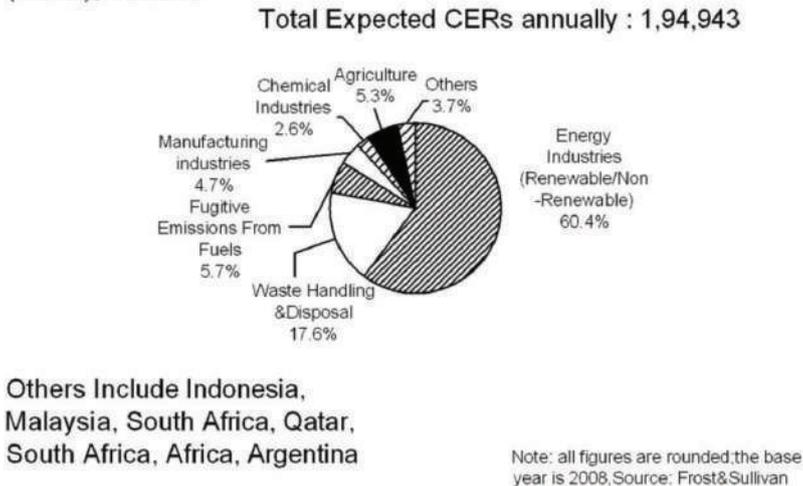Carbon Credits Market : Expected CERs from Registered Projects annually by Percent
(World), 2008-09

Total Expected CERs annually : 1,94,943

Others Include Indonesia,
Malaysia, South Africa, Qatar,
South Africa, Africa, Argentina

Note: all figures are rounded;the base
year is 2008,Source: Frost&Sullivan

Figure 5.63 Project wise carbon credit market as of year 2009

India has the second-highest carbon credit transacted volumes in the world. India is the pioneer in the biomethanation from STPs as demonstrated in the Dadar STP in Mumbai back in 1970s itself. The gas was directly piped to nearby institutions for their fuel and revenue was earned. The availability of duel fuel (diesel as well as biogas) as well as exclusive gas based engines came up subsequently in the 1980s and the generation plant was put up in Okhla STP in Delhi. With passage of time, the STPs have, by now perfected the technology of biomethanation and generate electrical energy and thus have accumulated the carbon credits.

Surat is perhaps the largest producer of biomethanation from STPs and treats about 600 MLD, out of which 3.5 MW of power is reportedly generated. This translates to 50,000 carbon credit units per year.

The magnitude of the carbon credits programme can be understood from the news article that India has bagged the world's largest carbon credit project that will help replace 400 million incandescent light bulbs with compact energy saving (CFL) bulbs at cheap prices in a year while preventing 40 million tonnes of carbon from entering the atmosphere annually.

The project will allow the government, investors, distribution companies (discoms) and CFL manufacturers to sell CFLs at Rs.15 each, instead of the Rs.100 they currently cost on an average, and have been approved by the UN under the global carbon credit scheme called Clean Development Mechanism. Thus, the need to plan STPs with inbuilt biomethanation and energy recovery is imperative. An illustrative comparison of the potential for energy recovery and the power actually needed for a STP with conventional ASP is shown in Table 5.28.

Table 5.28 Estimated and actual achieved electrical energy recovery

Design Flow	40 MLD
Primary Sludge - Suspended Solids	450 mg/l
Primary Sludge VSS as %	60%
SS Removal Efficiency	60%
Quantity of Primary Sludge	10890 kg/day
Percentage of VS in Primary Sludge	60%
Quantity of VS in Primary Sludge	6480 kg/day
Quantity of excess sludge at 0.35 kg/kg of BOD removed	3150 kg/day
Percentage of VS in Excess Sludge	85%
Quantity of VS in Excess Sludge	2677.5 kg/day
Total VS	9157.5 kg/day
VS Destruction Efficiency	55%
Bio-Gas Yield	0.75 Ncum/kg VS destroyed
Bio-Gas Generated	3777 m^3/day
Calorific Value	6.0 kWh/cum
Power Generated by Gas Engine	7931 kW/day
Power Required	7850 kW/day

Average of past two years
Bio-gas Generated   3200 m^3/day
Electrical Energy Generated   6600 kWh

It may be seen that as long as the designed sewage flow and the designed raw sewage BOD are available, the plant has the ability to not only be self-sufficient in power, but also capable of generating additional energy for nearby institutions and sell at an unfailing pattern and earn revenue as well. The caution needed at the time of design will however be, to opt for minimum of two gas engines and install only one to start with until the sewage quantities, qualities and biomethanation kinetics are established. After one engine is established, the second engine can be suitably sized to exploit extra power production if it becomes possible and make it a commercial proposition by feeding the local electricity grid, instead of drawing from it.

The 40 MLD conventional ASP at Chennai has been built with biogas utilization facilities by way of digester gas collection, wet scrubbing and dual fuel engine.

5.18   RECENT TECHNOLOGIES IN SEWAGE TREATMENT

There are many new technologies emerging for the treatment and reuse of sewage and sludge.

The objectives of sewage treatment are

(a) to metabolise the organic matter so as to produce an effluent which can be disposed in the environment without causing health hazards or nuisance and

(b) to produce a sludge which can be used as a soil filler if it comes out of a biological treatment or as a soil-sludge immobilized product like walkway paver blocks or compound wall bricks and thus preserve the environment from dumping waste products.

The discharge standards are formulated and specified for each case by the statutory authority of Pollution Control Board (PCB). Where there is reuse, the PCB requires a zero liquid waste discharge, which as far as the sludge is concerned, addresses its reuse or secure landfill.

Thus, the degree of treatment is dictated by the type of end use and the chemical characteristics specific to these.

Thus, there cannot be a fixed type of treatment technology for a reuse situation even within the same category of industries or situations. It is here that the emerging trends in technology play a crucial part and come in handy.

For example, an industry in a land locked area may find a roof top MBR as the best choice in view of its most precious land area at ground level which it can use for increasing the production, whereas for a municipality, such a concept does not arise in the first place.

However, even for the publicly funded STPs, the recent trend is to include nutrient removal and this has brought up a recent trend in extending well beyond conventional secondary treatment.

Thus, it is necessary to recognize and understand the emerging trends. At the same time, it needs to be also recognized that an emerging trend in India might have long been a standard trend elsewhere. At the same time, India cannot plumb for a trend of technology merely because it is in use elsewhere, especially in view of the introductory remarks in this Chapter 5 explaining why such blanket adoptions need local validation.

Thus, for the purpose if this chapter, any technology that is working successfully for more than 5 years on the same scale or larger in other countries, than at which it is intended to be used here in India and has potential to be sustainable in the Indian context, is considered as emerging trends in this section.

5.18.1 Objective Oriented Emerging Technologies

The type, objectives, process name, and outline of new sewage treatment technologies are summarised in Table 5.29 overleaf.

Table 5.29 Types of sewage treatment technologies

Type	Objective	Process	Description
Tertiary	Nitrogen (N) removal	Recycled biological Nitrification/ Denitrificatio n process	Concurrent with carbonaceous BOD removal nitrification is achieved by additional oxygen input and the mixed liquor and return sludge are recycled first into an anoxic tank receiving the raw sewage where the nitrates are denitrified by the microbes and partial BOD removal and almost complete removal of nitrates are obtained thus eliminating nitrogen altogether.
		Wuhrmann Process	This is a single sludge nitrification system with a downstream anoxic reactor. The influent enters into the aerobic tank where nitrification develops together with BOD removal and the nitrified mixed liquor passes to the anoxic reactor where the sludge is kept in suspension by moderate stirring, but no aeration. The denitrification takes place by microbes in their quest for oxygen. The classical difficulty here is the food requirements of the microbes which has otherwise been already removed in the aerobic reactor and hence, this process is suited only for sewages with little incoming nitrogen.
		Step-Feed Multistage Biological Nitrogen Removal Process	This process has been developed to enable efficient nitrogen removal from sewage without a major renovation The unit which consists of an anoxic tank and an oxic tank is arranged in several serial stages. The primary effluent is split and fed equally into an anoxic tank of each stage, and BOD load to MLSS in each stage is equalized. This method enables efficient nitrogen removal and easy maintenance of the process. The nitrified liquor internal recirculation from oxic to anoxic tank is carried out as needed.

Type	Objective	Process	Description
		Ammonia oxidation	Chlorination to mono, di & tri chloramines eventually releases nitrogen gas into atmosphere. It is a simple process, but the storage and handling of chlorination are a real challenge.
		Ammonia Stripping	Ammonia in raw sewage is present as ammonium carbonate and is a dissolved salt. By raising the pH to near 9.3, this is de-ionized and the ammonia becomes dissolved ammonia gas. It is then stripped in counter current towers where the sewage is sprayed from the top and air is blown from the bottom whereby the three phase mass transfer takes place and the resulting air ammonia gas mixture rises into the plume. The air volume and plume velocity are adjusted to keep the released ammonia concentration within threshold limits .
		Ion Exchange	Clinoptinolite is a resin occurring in some parts of the world which has the ability to exchange the ammonium ion and has applications in very small units needing cleaner operations and where secure landfill of spent resin is possible.
	Phosphorus (P) removal	Anaerobic-Oxic Activated Sludge Process	Under controlled conditions of anaerobiasis as absence of oxygen, luxury phosphorous uptake by the microbes is reported, but the mechanism is not fully understood. However, simultaneous denitrification can also occur from return activated sludge. Process operation with a plug flow regime & non-aerated but gently mixed zone in the first part of the activated sludge are stated as critical conditions to remove considerable phosphorus. However, the process control is influenced by varying phosphorous concentrations and sometimes it may be necessary to chemically precipitate the residual phosphorous by chemical addition.

Type	Objective	Process	Description
		Chemical precipitation	Aluminium and/or Ferric salts precipitate the phosphorous. This is easy to control.
		Bardenpho process	In this four-stage process, the first two stages are identical to the Modified Ludzck Ettinger (MLE) process of an anoxic zone followed by an aeration zone with a nitrate-rich recycle from the aeration to the anoxic zone. The third stage is a secondary anoxic zone for further denitrification to the portion of the flow that is not already recycled in first two-stages. A source like Methanol may be added to this third stage for microbial carbon. The fourth and final is a re-aeration zone to strip any nitrogen gas and increase the DO concentration.
	(N) & (P) Simultaneous removal	Anaerobic-Anoxic-Oxic Process	This process is a combination of the biological phosphorus removal process and the biological nitrogen removal process. It consists of tanks arranged in the sequence of anaerobic tank, anoxic tank and oxic tank. Influent and return activated sludge flow into the anaerobic tank while nitrified liquor is recycled with a circulating pump from the oxic (nitrification) tank to the anoxic (denitrification) tank. Ammonia nitrogen is oxidized to nitrite or nitrate in the oxic tank, and then nitrite or nitrate is denitrified to nitrogen gas in the anoxic tank. Aluminium and Ferric salts are added to chemically utilize the ionized Aluminium or iron & precipitate the phosphorous. This is a straightforward one for control.
Secondary	Polishing BOD, SS, Pathogen removal	Soil aquifer treatment	The soil organisms bring about further microbial activity is polishing these parameters and is a slow long term process. It is unique to the soil conditions, climatology, precipitation, flooding and inundation that may bring in agricultural residues of insecticides & pesticides.

Type	Objective	Process	Description
		Sequencing Batch Reactor (SBR)	This process utilizes a fill-and-draw reactor with complete mixing during the batch reaction step (after filling) and where the subsequent steps of aeration and clarification occur in the same tank. All SBR systems have common steps as (1) fill, (2) react (aeration), (3) sedimentation, (4) decant and (5) idle. For continuous-flow applications, at least two SBR tanks are provided in parallel so that one receives the sewage while the other completes its treatment cycle. Several process modifications have been made in the time durations associated with each step to achieve nitrogen and phosphorus removal.
		Moving Bed Biofilm Reactor (MBBR)	This process is based on several synthetic biofilm carrier elements (patented and non-patented) developed for use in the aeration tank of the ASP and are suspended in the activated sludge mixed liquor in the aeration tank. These processes are intended to enhance the activated sludge process by providing a greater surface area to unit volume of the aeration tank for the additional surfaces for increased microbial population and metabolism and hence, the biomass concentration in the tank and offers the potential to reduce the basin volumes. They are used to improve the volumetric nitrification rates and to accomplish the denitrification in aeration tanks by anoxic zones within the biofilm thickness.
		Fixed bed bio film activated sludge process (FFASP)	This process consists of a series of aerated reactors, filtration units and final polishing units. Plants with extensive root systems are placed on a supporting mesh slightly below the water level in the open aerated reactors and their roots dangling about 1.5 metres into the sewage is claimed to provide a healthy habitat for bacteria and a whole range of other organisms such as protozoa, zooplankton, worms, snails and even fish. As sewage flows through these different ecosystems in each tank, a series of self managing, cascading ecologies are stated to provide a highly robust and efficient system.

Type	Objective	Process	Description
		Immobilized Biofilm Process (Eco Bio Block, EBB)	This process is used for cleaning up polluted water sources such as sewage drains, polluted rivers, ponds, lakes, etc. The blocks are produced by mixing effective microbes with zeolites (volcanic porous stones), and alkaline cement. Once EBB is placed in polluted water, the effective microbes would multiply, treat the wastes effectively in a faster manner and clean the water body without causing any harm to plants and fish. EBB does not require energy, manpower and maintenance to perform the cleaning process. There is no operational cost practically. Since it is an online treatment, additional large land space is not required. However, the hydraulics of the drains needs validation with EBB in place
Advanced processes	Minute Solids in sus-pension colloidal materials, dissolved organic matter, TDS microbes, etc. removal	Membrane filtration	Membrane Filtration is used to remove minute solids, colloidal material, dissolved organic matter, etc. from secondary effluents using several kinds of membranes. According to separating particles size, membranes are classified as follows: Micro filtration (MF) for 0.08 to 2.0 microns Ultra filtration (UF) for 0.005 to 0.2 microns Nano filtration (NF) for 0.001 to 0.01 microns Reverse osmosis (RO) for 0.0001 to 0.001 microns The RO membrane is essentially a desalination adaptation for removal of TDS
		Membrane Bioreactor	This process is a sort of aeration tank and secondary clarifier being a two in one. The secondary clarifier is avoided by filtration of mixed liquor by membrane modules either immersed into the aeration tank mixed liquor or externally fitted and the mixed liquor routed through these. Essentially it has a suspended solids free treated sewage and retains higher MLSS and reduces the volume of aeration tanks.

Note: These are not a comprehensive listing as many newer processes keep evolving at any given time. However, in respect of biological processes, the descriptions above are meant for understanding the basics of the stated processes.

5.18.2 Recycled Nitrification / Denitrification Process

5.18.2.1 Description

Single-stage systems are those in which nutrient removal is achieved in a single basin and clarifier. Removal of nitrogen is achieved by combined nitrification (under aerobic conditions) and denitrification (under anoxic conditions). A single-stage system using one anoxic zone can achieve an effluent total nitrogen concentration of 4 to 11 mg/l as nitrogen. This is commonly called as Modified Ludzack-Ettinger (MLE) Process. The aspects of this are as follows:-

a) Nitrification is influenced by temperature, pH, DO and toxic or inhibiting substances.

b) Nitrification is possible between 5 to 50ºC. The optimum range is between 25 to 35ºC.

c) Nitrification is possible between pH 6.5 to 8.0 and the optimum condition is around pH 7.2.

d) The recommended level of dissolved oxygen is 2 mg/L

e) In order to promote biological reaction and to prevent deposits of activated sludge organisms, mixers are to be installed in the anoxic tank. The schematic drawing is shown in Figure 5-64.
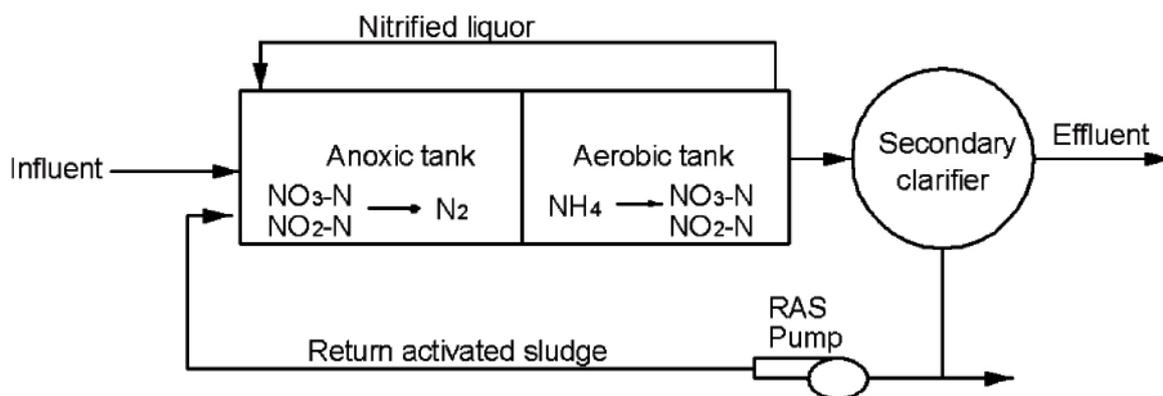

Figure 5.64 Configuration of recycled nitrification/denitrification process

5.18.2.2 Application Examples

The performance of full-scale recycled nitrification / denitrification process of Yoshinogawa STP, Yoshinogawa regional sewerage system of Nara Prefecture, Japan is shown in Tables 5-30 and 5-31.

Table 5.30 Operational conditions of Yoshinogawa STP, Japan (FY 2009)

Index	Influent Flow (m³/d)	HRT (hrs)		F/M	MLSS (mg/l)	Sludge Age (days)	Internal Recirculation (%)	Return Sludge (%)
		Anoxic	Oxic					
Average	50	3.3	8.4	0.083	2,130	17.0		51
Range	8,287-11,265	2.9-3.6	7.3-9.1	.058-0.122	,810-2,370	13.2-20.3	75	50-54

Source:Regional Sewerage Centre of Nara Prefecture, Japan, 2009

Table 5.31 Performance of Yoshinogawa STP, Japan (FY 2009

	BOD, mg/L			SS, mg/L			TN, mg/L		
Influent / Effluent / % Removal	173	3.3	98.1	186	6	96.8	28.6	7	75.5

Source: Regional Sewerage Centre of Nara Prefecture, Japan, 2009

5.18.2.3 The K&C Valley STP at Bengaluru

This is the first STP in India for BOD removal and biological nitrification and denitrification under JICA funding for Bangalore and functioning to its 30 MLD capacity. The criteria and performance are compared in Table 5.32 with the design criteria as in the often referenced textbook "Wastewater Engineering" by Metcalf & Eddy bringing out the need for similarly criteria for other projects as well.

Table 5.32 Design criteria for biological nitrification-denitrification from results of the 31 MLD average flow Koramangala & Chellagatta (K&C) Valley STP at Bengaluru

No.	Design Parameter	(A)	(B)
1.	Raw sewage BOD, mg/l	350	140
2.	Final BOD, mg/l	17	9
3.	Raw TKN, mg/l	37	35
4.	Final TKN, mg/l	8	6
5.	Raw sewage suspended solids, mg/l	400	70
6.	Final suspended solids, mg/l	26	10
7.	HRT in anoxic zone at average flow, hrs	1.6	2.5
8.	HRT in aeration zone at average flow, hrs	20.5	9
9.	MLSS in aeration zone, mg/l	3500	3000
10.	Average DO in aeration tank	2	2
11.	F/M in aeration	0.12	0.16
12.	Ratio of RAS to plant flow	1.0	0.6
13.	Ratio of MLSS return to plant flow	2.0	3.1
14.	Surface loading in clarifier at average flow in m^3/m^2/day	12	24
15.	HRT in clarifier at average flow + RAS, hrs	3.5	NG
16.	SWD of clarifier, m	3.5	NG
17.	Solids loading in clarifier kg MLSS/m^2/day	84	115
18.	Oxygen kg for kg BOD applied	1.2	1.48

(A)- Performance of K&C Valley STP at Bengaluru, (B)- As per Example 8-2 & 8-5 of Wastewater Engineering-Metcalf & Eddy-4th Edition, NG-Not Given

5.18.2.4   Advantages and Disadvantages

The advantages and disadvantages of this process are as follows:

a. Advantages

i.   As for usual municipal sewage, up to 85% of nitrogen removal can be expected in this process

ii. This process allows controlling the discharge of nitrogen to the receiving natural waters, which could create eutrophication problems. In this case the reduction of phosphorus is also required

iii. To limit the consumption of oxygen in the water bodies, because it requires approximately 4.57 mg of oxygen to oxidize 1 mg of nitrogen

iv. To facilitate the reuse of treated water in certain activities which require waters with low levels of nitrogen

b. Disadvantages

i.   Denitrification is possible only if a good nitrification has been achieved

ii. Nitrification is possible only if adequate bicarbonate alkalinity is available, otherwise bicarbonate alkalinity is to be added

iii. Precise control of floor level gentle mixing in anoxic tank and residual DO control in oxic tank are required

iv. Toxic and inhibiting substances may affect the activity of the nitrifying bacteria, this of course being common irrespective of the process used

5.18.2.5   Typical Design Parameters

The design criteria in Table 5.32 of the Koramangala & Chellagatta (K&C) Valley STP at Bengaluru is to be relied upon until further validations become available for Indian conditions.

5.18.2.6   Applicability

This process can remove 70% to 85% of the Total Nitrogen in the sewage. It is used for nitrogen control where inhibitory industrial waste is not present. It is easier to install in new plants and also upgrade existing ASPs as the additions are stand alone and are only a half hour anoxic tank, MLSS return and additional air supply.

5.18.3   Wuhrmann Process

5.18.3.1   Description

The Wuhrmann process configuration, as shown in Figure 5.65 (overleaf), is a single-stage nitrification system with the addition of an unaerated anoxic reactor. By this reason this process is also called post-denitrification.
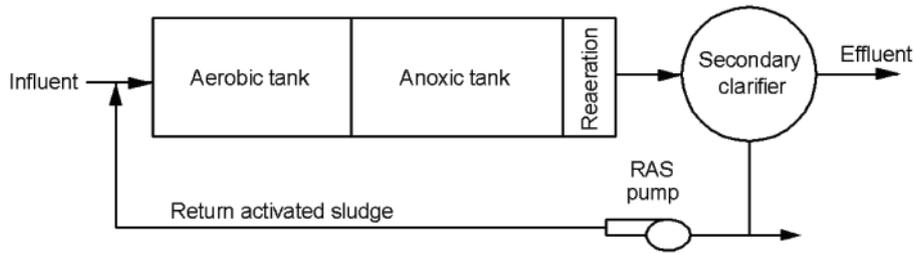
Figure 5.65 Configuration of Wuhrmann process

The possible lack of available carbonaceous substrate in the reactor significantly limits the denitrification rate of this configuration. To solve this problem an updated solution has been proposed by Ludzack and Ettinger in which the anoxic tank is the first in the sequence and is followed by the aerobic tank. Sludge from the secondary sedimentation is also recirculated to the inlet and mixed with the influent as shown in Figure 5-66. This process has a disadvantage in that a fraction of the nitrate generated in the aerobic tank is sent to the secondary sedimentation without denitrification.
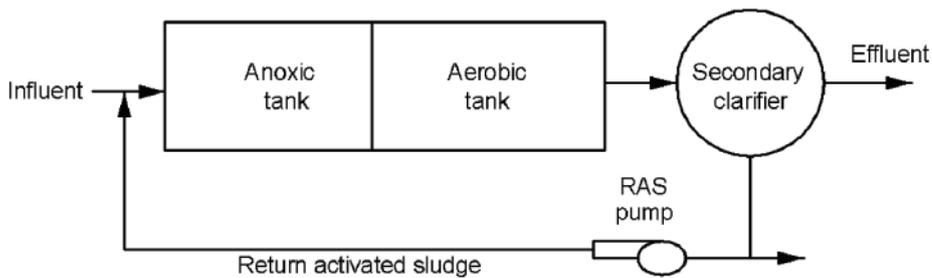
Figure 5.66 Configuration of Ludzack & Ettinger process – Wuhrmann modified

5.18.3.2   Application Examples

The process diagram and performance of full-scale Wuhrmann processes of Hamamatsu City sewage treatment plant, Japan are shown in Figure 5.67 and Table 5.33.
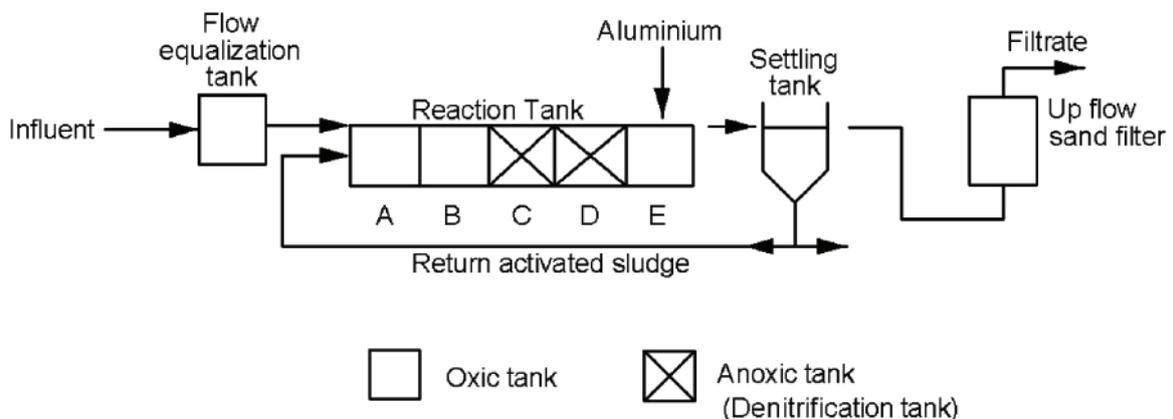
Source:Moriyama et al., 1988

Figure 5.67 Hamamatsu City sewage treatment plant process diagram

Table 5.33 Performance of Hamamatsu City sewage treatment plant

Parameter	Values	Parameter	Values
Water temperature (°C)	13.5 - 16.0	Influent	
Flow rate (m³/day)	919 - 1,008	BOD	100 - 400
Return sludge ratio (%)	86 - 105	T-N	30 - 50
Retention time (hours)	23.0 - 25.1	NH_4^+-N	20 - 30
Waste sludge (m³/day)	10.6 - 19.2	Effluent	
MLSS (mg/l)	3,670 - 3,880	BOD average	15
SRT (days)	22.7 - 37.6	T-N	5 - 10
BOD - SS loading (kg/kg/day)	0.15 - 0.23		
T N - SS loading (kg/kg/day)	0.028 - 0.037		
Sludge aeration time (days)	9.1 - 15.0		

Source:Moriyama et al., 1988

5.18.3.3   Advantages and Disadvantages

The advantages and disadvantages of this process are as follows:

a. Advantages

i. Process can virtually remove all nitrogen

ii. The influent enters into the first reactor, where nitrification develops, together with removal of almost all the biodegradable, organic material

iii. The nitrified mixed liquor passes to the second reactor, where the reduction of nitrate takes place

b. Disadvantages

i. In practice, re-aeration step is usually needed after anoxic zone

ii. For high N-removal, anoxic zone is very large

iii. The anoxic tank sometimes requires the addition of organic matter to allow denitrification. So, some organic matter in excess is added to the treatment process whose objective is to reduce the organic matter content

5.18.3.4   Typical Design Parameters

Typical criteria for nitrification-denitrification systems, Pre-denitrification (Modified Ludzack Ettinger process), Post-denitrification (Wuhrmann process), and Combined pre-and post-denitrification (Bardenpho process) are as in Table 5.34 (overleaf).

Table 5.34 Typical criteria for nitrification-denitrification systems (20°C MLSS temperature)

Parameter	Pre-denitrification	Post-denitrification	Combined Pre- and Post-denitrification
System SRT, days	6-30	6-30	8-40
HRT, hours			
First anoxic zone	2-8	-	2-6
First aerobic zone	6-12	6-12	6-12
Second anoxic zone	-	2-6	2-5
Reaeration zone	-	-	0.5-1.0
MLSS, mg/l	1500-4000	1500-4000	2000-5000
Nitrate recycle	2-4 Q*		2-4 Q
RAS flow	0.5-1 Q	0.5-1 Q	100% (Q design)
Dissolved oxygen, mg/l			
Anoxic zones	0	0	0
Aerobic zones	1-4	1-4	1-4
Mixing requirements			
Anoxic zones, kw/10^3 m^3	4-10	4-10	4-10
Aerobic zones, kw/10^3 m^3	20-40	20-40	20-40
Airflow, aerobic, m^3/min 10^3 m^3	10-30	10-30	10-40

* Energy input is an important parameter; however, the manufacturer should be consulted for determining the number and placement of mixers.
Propeller and turbine mixers have been used successfully.

Source:WEF, 2010

If there is no data for DO inf, then assume a value of 2mg/l at 15ºC or less; 1 mg/l at 15ºC to 20ºC; and 0.5 mg/l at temperatures greater than 20ºC. The DO_{NR} can be assumed equal to the dissolved oxygen of the mixed liquor in the vicinity of where the nitrate recycle pump is located. They may be equal to the dissolved oxygen at the end of the aerobic zone. The dissolved oxygen of the RAS is difficult to determine without sampling the RAS or the clarifier blanket. In the absence of any data, it may be assumed that it is half the dissolved oxygen level at the end of the aerobic zone. Calculate the nitrite-nitrogen and nitrate-nitrogen load entering the pre-anoxic zone (kg/d). The criteria about nitrification, denitrification and oxygen demand are shown below:

a. Total oxygen required, as part of organic material, is 4.57 mg O_2/mgN for nitrification of which 2.86 mg O_2/mgN is released in denitrification.

b. Denitrification reduces the oxygen demand by 23g O_2/hab/d (8×2.86).

c. For an assumed contribution of nitrogen of 10 g N/person/d and an estimated requirement for sludge production of 2 g N/person/d (i.e., 20% of the influent TKN), the nitrification potential is 8g N/person/d.

d. For complete denitrification, the nitrate mass to be denitrified equals 8 g N/hab/d.

e. If methanol is used as external organic material, the consumption is 2.5 g CH_3OH/g N.

f. The per capita contribution of nitrogen varies from 2 to 5g/p/d of organic nitrogen and 3 to 7g/p/d of ammonia. The contribution of NO_3 is negligible. However, industrial activity can contribute with big amounts of nitrogen compounds.

5.18.3.5   Applicability

The simple nitrification-denitrification systems shown in this section are one of the processes used for nitrogen removal in sewage treatment as an option to control eutrophication. These processes can remove 70% - 80% of the total nitrogen in the sewage.

5.18.4   Step-feed Multistage Biological Nitrogen Removal Process

5.18.4.1   Description

In step-feed process, the basin is divided into several stages and raw influent is introduced to each stage proportionately. All return micro-organisms (sludge) are introduced at the head of the basin.

By splitting the flow to several influent feed locations and directing recovered sludge to the beginning of the process, a higher solids retention time is achieved compared to plug-flow system with the same basin volume.

Common features include anoxic zones for denitrification, and oxic zones for oxidation of organic material, nitrification and phosphorus uptake. Nitrified mixed liquor is returned from the oxic zone to the anoxic zones for denitrification.

5.18.4.2   Application Examples

The application example of step-feed multistage biological nitrogen removal process of Stamford STP, Stamford, CT, USA is shown in Figure 5-68 overleaf.

The original STP was a traditional activated sludge system. It was updated to reduce nitrogen in the effluent to the bay.

The aeration tanks were divided in 4 phases with 25% each to operate with step feed. The incoming load was divided in 4 tanks and RAS was added in the first tank.

The yearly average discharge of TSS is 10mg/L and BOD is 6mg/l.

Nitrogen was reduced from 8-7.5mg/L to an average of 3mg/l.
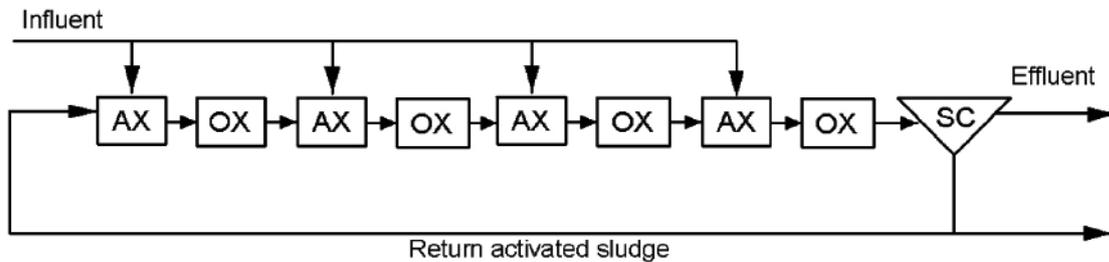
Source: Christine deBarbalillo et al., 2002

(AX - Anoxic, OX - Oxic, SC - Secondary Clarifier)

Figure 5.68 Step feed schematic for four pass anoxic-oxic system in Stamford

5.18.4.3 Advantages and Disadvantages

The advantages and disadvantages of this process are as follows:

a. Advantages

 i. Better equalization of the waste load

 ii. Operation flexibility

 iii. More uniform oxygen demand along the aeration tank, with lower peak demand

 iv. Allows operational control of the sludge age and hydraulic residence time

 v. Can be used in preventing gross process failure due to hydraulic overloading or sludge bulking

 vi. The sludge is reused several times down the tank, allowing for higher BOD treatment capacity

 vii. This design reduces aeration tank size and aeration time, while BOD removal efficiency is maintained. The shorter aeration time reduces capital expenses.

b. Disadvantages

 i. Requires good O&M of mechanical and control equipment to assure the right flow distribution and liquid recirculation

5.18.4.4 Typical Design Parameters

The main design criteria of this process are shown in Table 5-35 overleaf. The important considerations related to design of this process are as follows:

 a. Anoxic tank with DO 0.2 mg/L and Oxic tank with DO 2.0 mg/l.

 b. Denitrification Rate: 60% - 80%

 c. It is a compact design. With the reduction in hydraulic retention time, the tank capacity is reduced to half that of a conventional circulating process.

 d. It is energy-efficient. According to the need of nitrified liquid recirculation, energy consumption can be cut to about 70% of the usual recirculation process.

Table 5.35 Typical criteria for Step-feed multistage biological nitrogen removal process

SRT (days)	F/M as MLVSS	Volumetric Loading (kg BOD$_5$/m^3/d)	MLSS (mg/l)	HRT (V/Q) (hours)	Return Activated Sludge Qr/Q
5–15	0.2–0.4	0.64–0.96	2,000–3,500	3–5	0.25–0.75

Source: Metcalf & Eddy, 2003

5.18.4.5 Applicability

Existing facilities can be modified. Modification can be carried out without sacrificing existing treatment capacity, installing partitions in the reaction tank, creating anoxic and aerobic zones and adding distribution system for the step-feed of the primary effluent.

To be effective, all of the recirculated sludge needs to be returned to the front end of the tank, and not mixed with the influent sewage.

It addresses the common problem of filaments caused by high F/M at the front end of the tank and filaments caused by low F/M at the back end of the tank.

5.18.5   Anaerobic-oxic Activated Sludge Process

5.18.5.1   Description

Anaerobic-Oxic (Aerobic) activated sludge process is a biological phosphorus removal (BPR) process which removes phosphorus from sewage due to luxury uptake of phosphorus by activated sludge microorganisms. The schematic is shown in Figure 5.69.
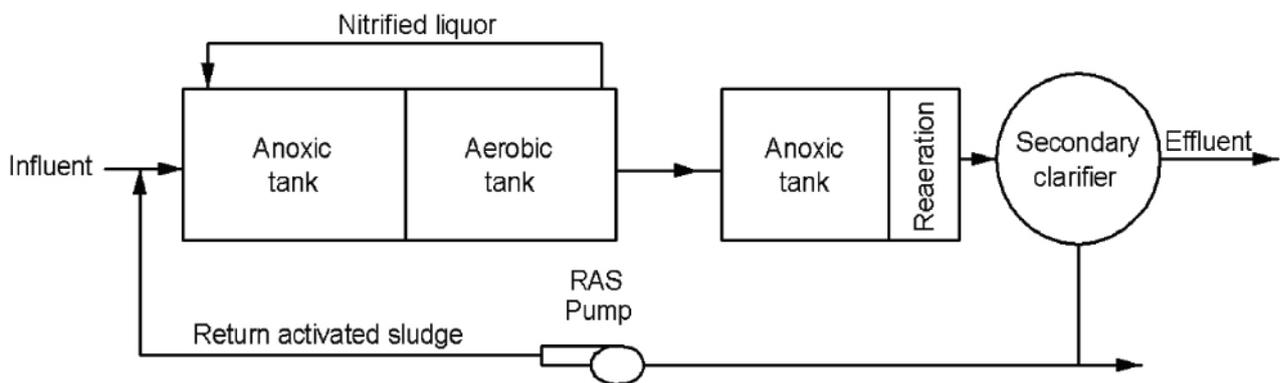

Figure 5.69 Configuration of anaerobic-oxic activated sludge process

Phosphorus is an important element of activated sludge microorganisms. Although the phosphorus content of activated sludge microorganisms is generally between 1.5 to 3%, it is possible to absorb phosphorus at a high concentration level between 2.5 to 5% by placing activated sludge under a special condition.

Specifically, activated sludge microorganisms release phosphorus into sewage in an anaerobic condition (where neither dissolved oxygen nor joint oxygen such as nitrate exists in water), and in an aerobic condition which follows the anaerobic condition, activated sludge microorganisms absorb more phosphorus than that emitted into sewage.

This phenomenon is called luxury uptake of phosphorus by activated sludge microorganisms.

This is an outstanding method for removing a considerable quantity of phosphorus from sewage without adding chemicals.

However, since the phosphorus removal efficiency of this system is affected by seasonal changes and climate such as rainfall, it is necessary to compensate for these changes by adding flocculating agent to tackle the in organic phosphorus.

5.18.5.2   Application Examples

Some examples of performance of the full-scale AO processes are shown in Table 5.36.

Table 5.36 Performance of AO processes

Site	HRT (hours)		MLSS (mg/l)	Total Phosphorous (mg/L)			Ref
	Anaerobic	Oxic		Influent	Effluent	Removal %	
Largo, USA	1.5	2.6	-	8.9	1.85	79.2	1
Pontiac, USA	1.8	6.7	2,430-2,820	3.0-4.1	0.4-0.8	80.0	2
Kawasaki JP	2.0	3.5	2,200	2.05-4.54	0.16-0.45	90.8	3
Fukuoka JP	3.3	9.7	1,620	8.2	0.63	92.3	3

Ref 1: USEPA, 1987
Ref 2: USEPA, 1987
Ref 3: Murata, 1992

5.18.5.3   Advantages and Disadvantages

The advantages and disadvantages of this process are as follows:

a.   Advantages

 i. For municipal sewage, about 80% of organic phosphorus removal efficiency can be expected.

 ii. It is known that this process can control bulking of activated sludge caused by filamentous bacteria in addition to phosphorus removal.

b. Disadvantages

i. When the organic matter concentration (BOD or COD) of influent is low on rainy days, etc., phosphorus contained in the activated sludge microorganisms is not adequately released to liquid phase in the anaerobic tank; therefore, the removal efficiency of phosphorus deteriorates. It is desirable to maintain BOD/P in excess of 20 to 25 for good phosphorus removal.

ii. This process is susceptible to side stream containing high concentration of phosphorus from a sludge treatment system.

5.18.5.4   Typical Design Parameters

According to the performance of the actual AO processes and references on the AO process, typical design parameters of this process are integrated as shown in Table 5.37.

Table 5.37 Typical design parameters of AO process

F/M as MLVSS	SRT (days)	MLSS (mg/l)	HRT(hours)		Return activated sludge (% of influent)
			Anaerobic	Oxic	
0.2-0.7	2-25	2,000-4,000	0.5-1.5	1-3	25-40

Source: Pennsylvania Department of Environmental Protection

Important considerations related to design and maintenance of this process are as follows:

a On rainy days, etc., the removal efficiency of phosphorus deteriorates in this process. To remove phosphorus in a stable manner, equipment for adding chemical coagulant or filtration equipment, etc., need to be installed as auxiliary equipment.

b. Division wall between aerobic and anaerobic tank must be installed to prevent the adverse current of the activated sludge mixed liquor as far as possible. Moreover, since scum occurs in the anaerobic tank, it is desirable to install equipment that eliminates the generated scum.

c. To promote biological reaction and to prevent deposition of activated sludge, mixers are to be installed in the anaerobic tank.

d. The structure of the primary and final sedimentation tank should be the same as that used in a conventional activated sludge process.

e. In a sludge treatment facility, when waste activated sludge remains in the anaerobic condition, phosphorus contained in activated sludge microorganisms is released to the liquid phase.

Therefore, it is necessary to take measures to prevent such a phenomenon so that the phosphorus concentration in treated water does not increase in response to the influence of phosphorus load of the side stream.

5.18.5.5   Applicability

This process can treat BOD and SS of municipal sewage to a level equivalent to that of conventional activated sludge process, and the phosphorus removal efficiency of the A/O process depends primarily on the ratio of the BOD concentration to the phosphorus concentration in the influent. Effluent soluble phosphorus concentrations as low as 1 mg/L are possible when this ratio exceeds 10:1.

5.18.6   Bardenpho Process

5.18.6.1   Description

The first two stages of the four-stage Bardenpho process are identical to the Modified Ludzack Ettinger (MLE) system (anoxic zone followed by an aeration zone with a nitrate-rich recycle from the aeration to the anoxic zone). The third stage is a secondary anoxic zone to provide denitrification to the portion of the flow that is not recycled to the primary anoxic zone. Methanol or another carbon source can be added to this zone to enhance denitrification. The fourth and final zone is a re-aeration zone that serves to strip any nitrogen gas and increase the DO concentration before clarification. This process can achieve effluent TN levels of 3 to 5 mg/L (USEPA, 2009). The schematic is shown in Figure 5.70.
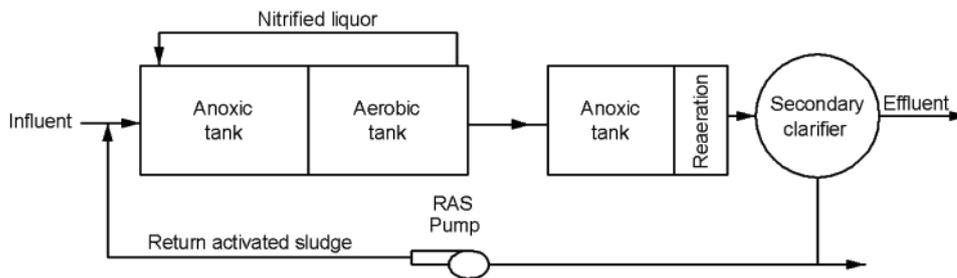

Figure 5.70 Configuration of Bardenpho process (4-stage)

The four stage Bardenpho process can be modified for the five stage system to remove both nitrogen and phosphorus. This system provides anaerobic, anoxic, and aerobic stages for phosphorus, nitrogen, and carbon removal. A second anoxic stage is provided for additional denitrification using nitrate produced in the aerobic stage as the electron acceptor, and the endogenous organic carbon as the electron donor. The final aerobic stage is used to strip residual nitrogen gas from solution and to minimize the release of phosphorus in the final clarifier. Mixed liquor from the first aerobic zone is recycled to the anoxic zone. The schematic is shown in Figure 5.71.
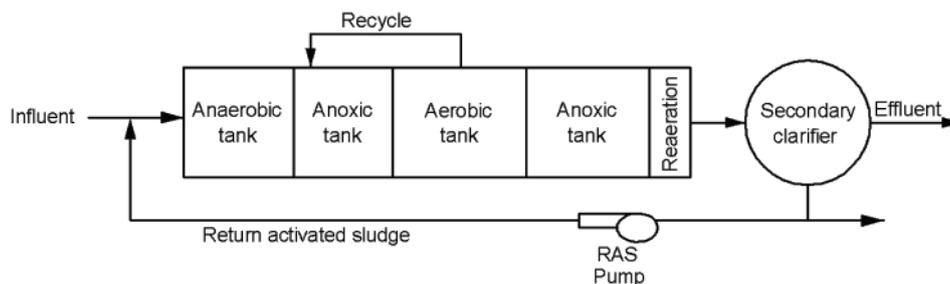

Figure 5.71 Configuration of modified Bardenpho process (5-stage)

5.18.6.2 Application Examples

The performance of full-scale Bardenpho process of 42 MLD Kelowna STP, the City of Kelowna, Canada is shown in Table 5.38.

Table 5.38 Performance of 42 MLD Bardenpho process, Kelowna STP, Canada

Parameter	Unit	Influent	Effluent
TSS	mg/l	350.0	2.0
BOD$_5$	mg/l	186.0	5.0
T-P	mg/l	8.50	0.14
T-N	mg/l	36.39	4.03
Faecal Coliform	cfu/100ml	N/A	0.2

Flow Capacity: 42 MLD

Process configuration: Bardenpho process + Dual media filtration process + UV disinfection process

Source: Environmental Operators Certification Program http://www.eocp.org/plants-kelowna.html

5.18.6.3 Advantages and Disadvantages

The advantages and disadvantages of this process are as follows:

a. Advantages

  i. The Bardenpho process removes nitrogen to low concentrations.

  ii. Addition of an anaerobic zone at the beginning of the process enables phosphorus removal.

  iii. Since the nitrates in the RAS ranges from 1 to 3 mg/L, it does not seriously interfere with the mechanism for phosphorus removal as can happen in the 3 Stage Pho-redox process.

b. Disadvantages

  i. Larger reactor volumes are required.

  ii. Precise control of anaerobic, anoxic, and oxic conditions in the tanks is required.

  iii. Toxic and inhibiting substances may affect the activity of the nitrifying bacteria.

5.18.6.4 Typical Design Parameters

Typical design parameters are shown in Table 5.39 overleaf.

5.18.6.5 Applicability

This process can achieve 3-5 mg/L T-N in unfiltered effluent.

Table 5.39 Typical design parameters of Bardenpho process

Process	SRT (d)	MLSS (mg/L)	Retention time (hrs)				Return Activated Sludge % of influent (%)	Internal Recycle % of influent (%)
			Total	Anaerobic	Anoxic	Aerobic		
Bardenpho (4-stage)	10-20	3,000-4,000	8-20	-	1-3 (1st stage) 2-4 (3rd stage)	4-12 (2nd stage) 0.5-1 (4th stage)	50-100	200-400
Bardenpho (5-stage)	10-20	3,000-4,000	9-22	0.5-1.5	1-3 (2nd stage) 2-4 (4th stage)	4-12 (3rd stage) 0.5-1 (5th stage)	50-100	200-400

5.18.7 Anaerobic-anoxic-oxic (A2O) Process (Biological Nitrogen and Phosphorus Removal Process)

5.18.7.1 Description

This process consists of tanks arranged in the sequence: anaerobic tank, anoxic tank and oxic tank. Influent and return activated sludge flow into the anaerobic tank while nitrified liquor is recycled with a circulating pump from the oxic (nitrification) tank to the anoxic (denitrification) tank.

Ammonia nitrogen is oxidized to nitrite or nitrate in the oxic tank, and then nitrite or nitrate is denitrified to nitrogen gas in the anoxic tank. Depending on the water quality of influent to the reaction tank, it may be necessary to add organic matter such as methanol and sodium hydroxide etc., to the reaction tank. Coagulant may be added to the reaction tank if more stable phosphorus removal is needed. The schematic is shown in Figure 5.72
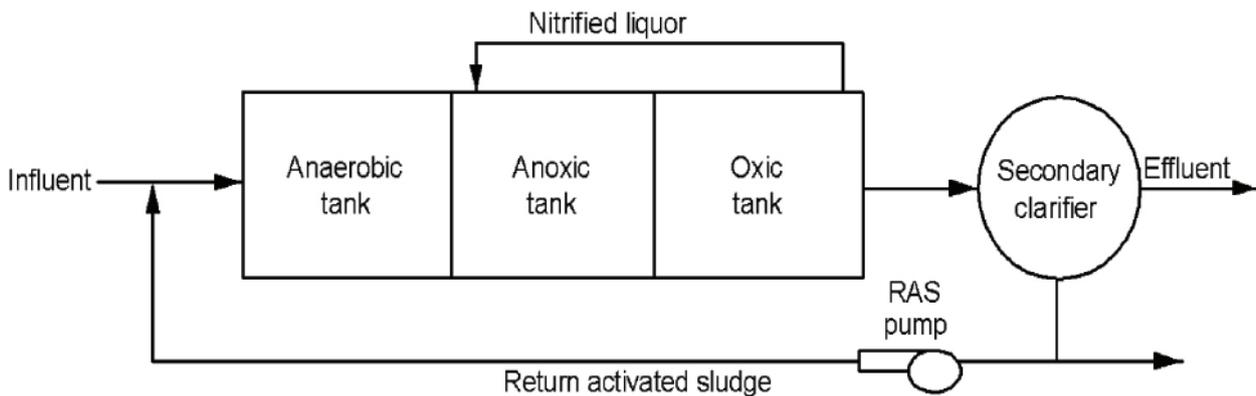

Figure 5.72 Configuration of anaerobic-anoxic-oxic activated sludge process

5.18.7.2 Application Examples

The performance of full-scale A2O process of Toba STP in Japan is shown in Table 5 40 overleaf.

Table 5.40 Performance of A2O process (Toba STP, Kyoto, Japan)

Run	Water Temp. (°C)	HRT (hr) Anaerobic	Anoxic	Oxic	F/M	MLSS (mg/l)	TN or TP	Influent mg/l	Effluent mg/l	Removal (%)
1	17.6	1.3	2.7	6.0	0.12	1,570	TN	27.0	8.3	69
							TP	2.7	0.8	70
2	14.9	1.3	2.6	6.0	0.14	1,560	TN	26.9	10.0	63
							TP	2.8	1.1	61
3	20.6	1.3	2.7	6.1	0.12	1,810	TN	24.7	8.9	64
							TP	3.0	0.3	90
4	23.8	1.3	2.6	5.8	0.08	1,930	TN	17.5	6.9	61
							TP	1.9	1.5	21

Source: Murata, 1992

5.18.7.3   Advantages and Disadvantages

a. Advantages

i. Both nitrogen and phosphorus are removed simultaneously in this process.

ii. A portion of alkalinity consumed in the aerobic tank is recovered by denitrification reaction in the anaerobic reaction tank by recycling nitrified liquor from the aerobic tank to the anoxic tank.

b. Disadvantages

i. Generally, this process needs larger volume of reaction tank than that used in the standard activated sludge process.

ii. The process operating parameters of nitrogen removal, such as SRT conflict with that of phosphorus removal; therefore, the optimum SRT condition needs to be set to remove both nitrogen and phosphorus. Generally, the phosphorus removal efficiency is less than that of the AO process because higher SRT value is needed than in the AO process.

5.18.7.4   Typical Design Parameters

According to the performance of the actual A2O processes and references on the A2O process, typical design parameters of this process are integrated as shown in Table 5.41.

Table 5.41 Typical design parameters of A2O process

F/M as MLVSS	SRT (days)	MLSS (mg/L)	HRT(hours) Anaerobic	Anoxic	Oxic	Return activated sludge (% of influent)	Nitrified Recycle (% of influent)
0.15-0.25	4-27	3,000-5,000	0.5-1.5	0.5-1.0	3.5-6.0	20-50	100-300

Source: Pennsylvania Department of Environmental Protection

5.18.7.5   Applicability

For municipal sewage of ordinary water quality, the nitrogen removal efficiency of this process is expected to be 60 to 70% and phosphorus removal efficiency is expected to be 70 to 80%.

5.18.8   Other Reported Processes

5.18.8.1   Sharon-Anammox Processes

The acronyms Sharon is from "Single High rate Ammonia Removal over Nitrite" and Anammox is from "Anaerobic Ammonia Oxidation.". In this process, it is stated that a portion of the incoming ammonia is partially oxidized to nitrite and the balance ammonium is retained. These are subsequently converted into nitrogen gas under anaerobic conditions without the need to add an external carbon source. It is claimed that 40% less oxygen is adequate as compared to conventional biological nitrification, organic carbon source is not required, and sludge production is negligible. Its development into a functional plant is however, still in its nascent stage. A schematic of nitrogen transformations is in Figure 5.73.
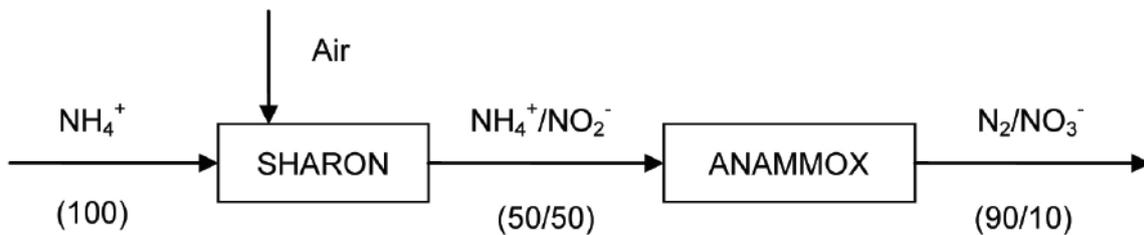

Figure 5.73 Schematic of Sharon-Anammox process for nitrogen removal

5.18.9   Guiding Principles of Biological Nutrient Removal Processes

Guiding principles of the biological nutrient removal processes which are described in the foregoing sub-sections are summarised as under:

a. Make sure whether nitrogen and phosphorous removal is required as per the discharge standards or the sewage and quality standards.

b. Decide whether removal of N or P or both is required.

c. All the design criteria are applicable for developed countries where detergents used are non-phosphate (biological) detergents. Check whether design criteria are directly applicable to India or not, where soaps and detergents containing chemical phosphates are commonly used.

d. Arising out of this, conduct a pilot-scale study to evolve guidelines that are to be followed especially for phosphorous.

e. If laboratory study is difficult, chemical removal (precipitation) of phosphorous by lime or alum is practicable.

f. In India, large scale STPs using such technologies have already been provided by the BWSSB in their STPs under the JBIC funding and these can be referred upon to draw factual data on design criteria vis a vis actual performance. Here again, this design is to be taken only as a guide and not replicated because biological nutrient removal is a complex process and has to be specific to the situation on hand based on BOD, nitrogen and phosphorous variations.

5.18.10 Membrane Filtration (MF, UF, NF, RO)

5.18.10.1 Description

Filtration membranes are classified according to the pore size or the size of solute they screen out as shown in Figure 5.74:
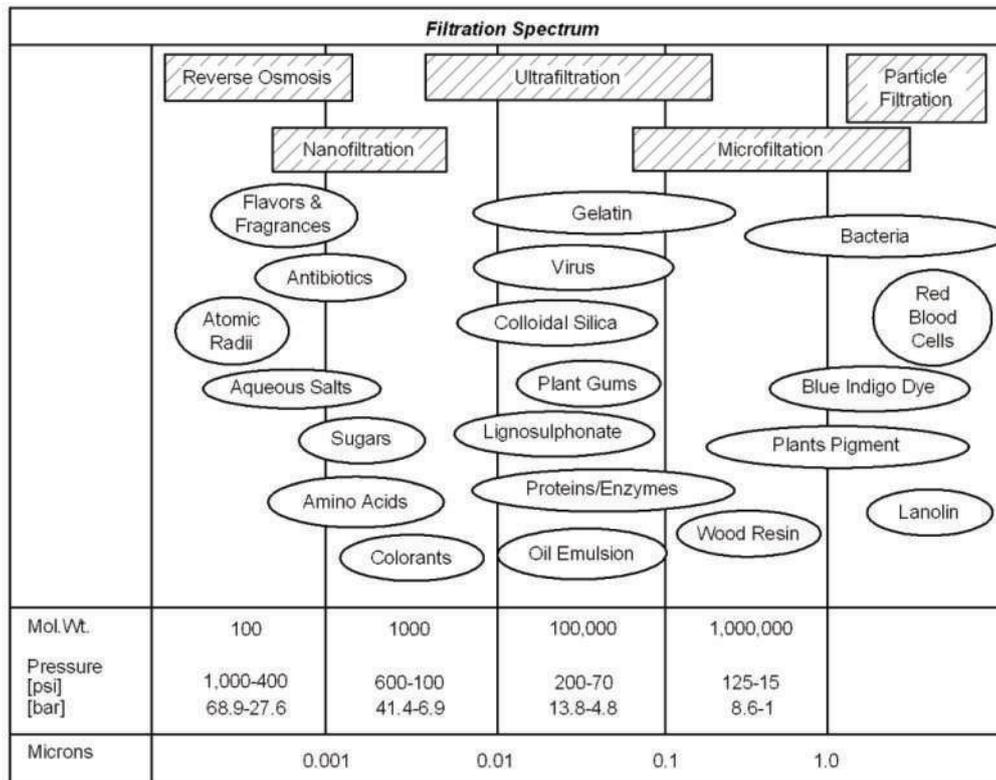

Figure 5.74 Membrane characteristics

MF - Microfiltration membranes are porous membranes with pore sizes between 0.1 and 1 micron (1 micron=1000 nanometre). They allow almost all dissolved solids to get through and retain only solids particles over the pore size.

UF - Ultrafiltration membranes are asymmetric or composite membranes with pore sizes around between 0.005 and 0.05 micron. They allow almost mineral salts and organic molecules to get through and retain only macromolecules

NF – Nanofiltration membranes are reverse osmosis with pore sizes around 0.001 micron. They retain multivalent ions and organic solutes that are larger than 0.001 micron.

RO - Reverse osmosis membranes are dense skin, asymmetric or composite membranes that let water get through and reject almost all salts.

5.18.10.2   Advantages and Disadvantages

The advantages and disadvantages of this process are as follows:

a. Advantages

  i. Membranes can reduce contaminants to the levels required by specific reuse application.

  ii. Membranes may be added at any moment in existing sewage treatment plant and may be designed for the amount of treated water needed

b. Disadvantages

  i. This process is only possible after appropriate treatment.

  ii. High operation and maintenance costs

  iii. Part of the water produced must be used for backwash

  iv. Backwash water must be incorporated to the influent water

  v. In case of RO, disposal of rejects with high dissolved solids is a problem, unless coastal discharge is available.

5.18.10.3   Typical Design Parameters

The appropriate combination of feed flow rate and TMP (Transmembrane Pressure) will maximize the flux while minimizing the impact of pumping and shear on the product. The appropriate combination of these two parameters will also minimize processing time and/or membrane area.

Membrane Area [m^2] = Process Volume [L] / (Flux [LMH]   Process Time [h])

Pump feed rate [L/min] = Feed flux [L/min/m^2]   Area [m^2]

LMH: L/m^2/h

These are fixed by membrane manufacturers.

5.18.10.4   Applicability

Membrane filtration is used for polishing water for specific uses like industry process water, or for aquifer infiltration.

In India, membrane filtration is widely used in the water and wastewater sectors. The recent sewage reclamation plants in India have all used membrane filtration to recover usable grade water from sewage in the final filtration section. It is used in various industries, airport complexes, and so on.

The difference between the UF, NF and RO is essentially the particle size removal since they depend on the pore size and colloidal particles can be removed by ultra filtration whereas dissolved salts will require reverse osmosis.

5.18.10.5   Guiding Principles

Membranes should be recognized more for the reject streams they generate and which are concentrated in pollutants. Unless it is addressed, the membrane technology will not be useful.

Spiral wound membranes need pre-treatment for complete elimination of SS, limiting COD to less than 100 mg/l and silt density index to less than 3 units besides elimination of all organics and refractory organic colour which are extremely important and complete sterilization of the feed which are challenges not easily surmountable.

Tube and disc arrangement of membranes, where the fluid passes over only one membrane film at a time and drains out in upward movement is relatively better as it can take feed with BOD, COD, SS etc. but these are costlier than spiral wound membranes.

5.18.11   Membrane Bioreactor

5.18.11.1   Description

The membrane bioreactor (MBR) process is a combination of activated sludge process and membrane separation process. Low pressure membranes (ultrafiltration or microfiltration) are commonly used. Membranes can be submerged in the biological reactor or located in a separate stage or compartment and are used for liquid-solid separation instead of the usual settling process. Primary sedimentation tank, final sedimentation tank and disinfection facilities are not installed in this process. The reaction tanks comprise an anoxic tank and an aerobic tank, and the membrane modules are immersed in the aerobic tank. Pre-treated, screened influent enters the membrane bioreactor, where biodegradation takes place. The mixed liquor is withdrawn by water head difference or suction pump through membrane modules in a reaction tank, being filtered and separated into biosolids and liquid. Surfaces of the membrane are continuously washed down during operation by the mixed flow of air and liquid generated by air diffusers installed at the bottom of the reaction tank.

The permeate from the membranes is the treated effluent.

The schematic is shown in Figure 5-75 overleaf.

5.18.11.2   Application Examples

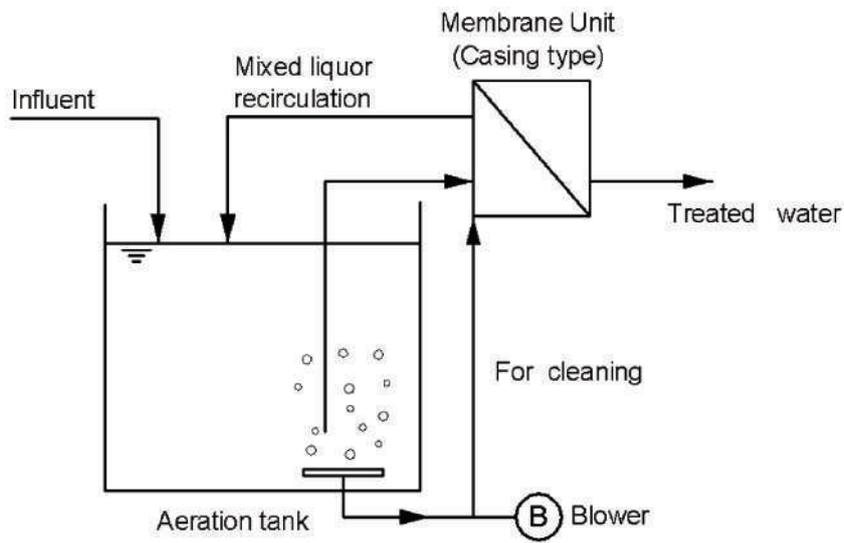**a. 4.54   MLD STP using MBR technology in the Games Village Complex, Delhi**

The schematic flow diagram of 4.54 MLD STP using MBR technology in the Games Village Complex is shown in Figure 5.76 and the description of each process of this plant are shown in Table 5.42 The plant data are as follows:

 • Average flow: 4.54 MLD
 • Peak Flow: 11.35 MLD
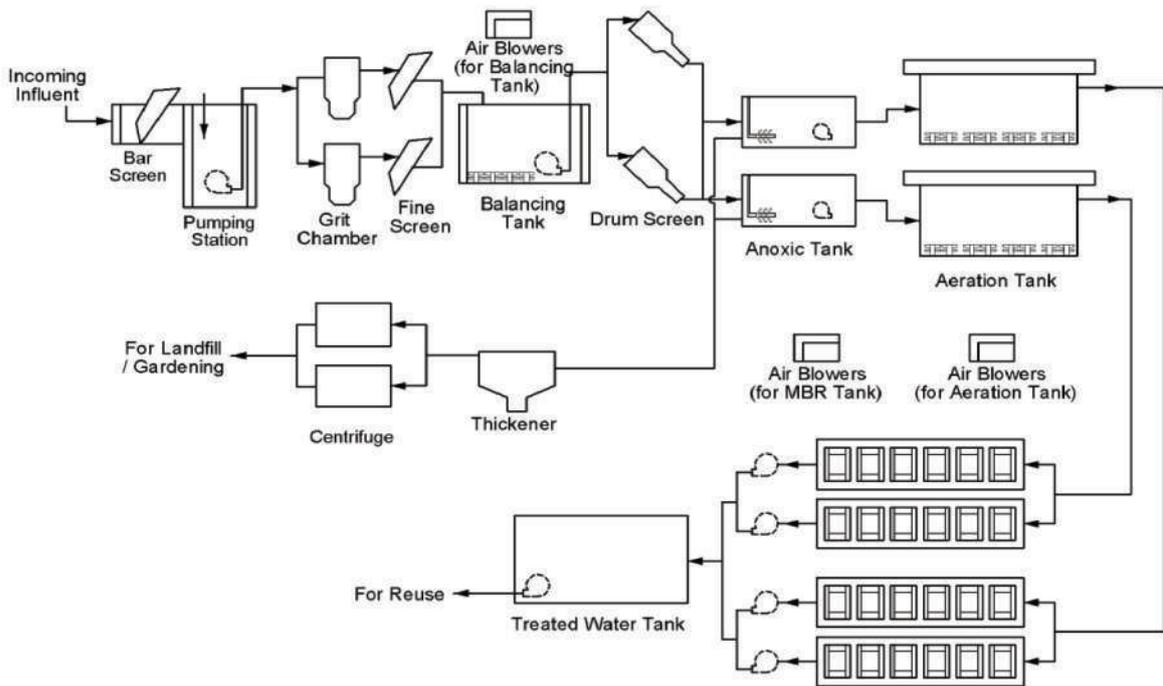 • Lean Flow: 2.0 MLD
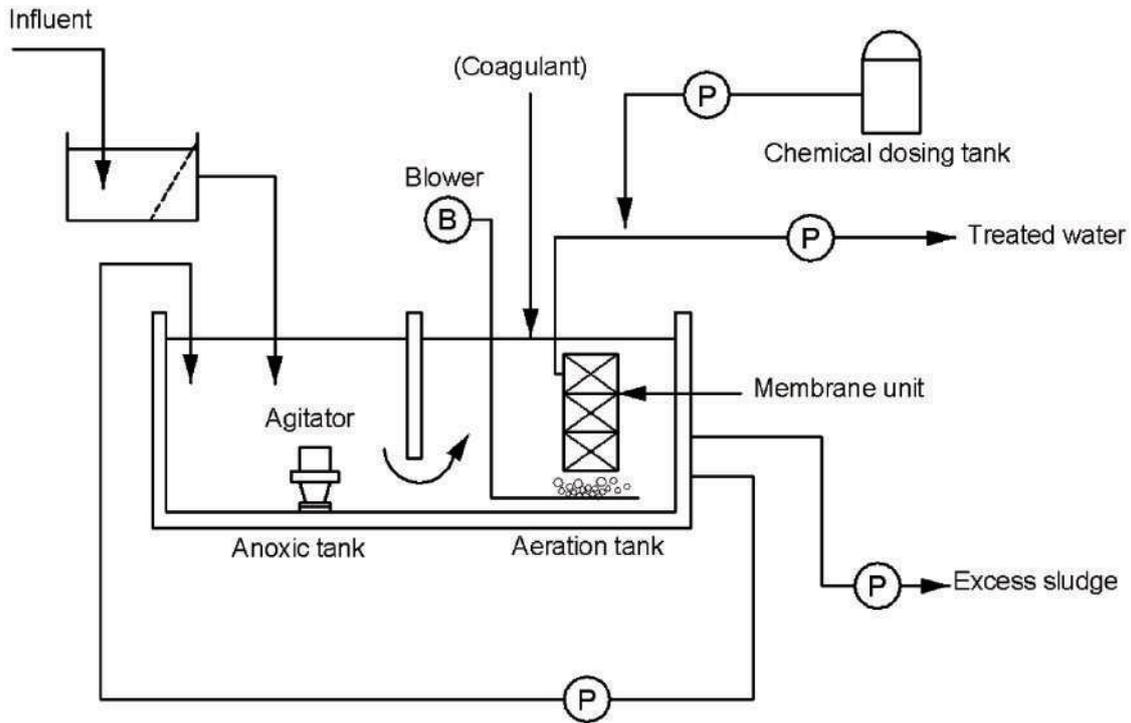
(a) Immersed membrane system

(b) External membrane system

Source:JSWA, 2003

Figure 5.75 Configuration of membrane bioreactor system

Source: Pamphlet issued by DJB

Figure 5.76 The schematic flow diagram of 4.54 MLD STP using MBR technology, Delhi

Table 5.42 Description of each process

Bar Screen	Fine Screen	Equalization/ Balancing Tank	Ultra Fine Screen	Anoxic Tank	Aeration Tank	Membrane Bio Reactor	Treated Water Holding Tank
1 Nos.	1 Nos.	1 Nos.	2 Nos.	2 Nos.	2 Nos.	4 Nos.	1 Nos.
Opening 20 mm	Opening 6 mm	2,900 m^3 (HRT 6 hrs)	Opening 1 mm,	370 m^3 (Total) (HRT 1.9 hrs)	830 m^3 (Total) (HRT 6.5 hrs)	473 m^3 (Total)	2,244 m^3

Source: Pamphlet issued by DJB

The performance of the MBR based STP at the Games village complex in Delhi is mentioned in Table 5.43 overleaf.

b. Nordkanal Sewage Treatment Plant, Germany

The operating conditions and the performance of full scale MBR process in Germany (Applied for the recycled nitrification-denitrification process) are shown in Table 5.44 and Table 5.45 overleaf.

The average flow of the MBR process is 1,024m^3/h.

Table 5.43 Performance of 4.54 MLD STP using MBR technology, Delhi

Parameter	Unit	Influent	Effluent
pH	-	7.0 - 7.6	6.8 – 7.8
Temperature	°C	18 - 38	-
TSS	mg/l	400	<1.0
BOD5	mg/l	250	<2
Total COD	mg/l	750	-
Total Kjeldahl Nitrogen	mg/l	40	<1.0
Total NH_4^+-N	mg/l	25	<0.5
Total Alkalinity	mg/l	305	-
Total Coliform	MPN/100 ml	-	2

Source: Pamphlet issued by DJB

Table 5.44 Operating conditions of MBR process (Nordkanal STP)

F/M	HRT(hours)		SRT (days)	MLSS (mg/l)	Flux (l/sqm/day)	Nitrified Recycle (% of influent)
	Anoxic	Oxic				
0.04	3.1-4.9	4.8-7.6	25.0-28.6	9,200	288-552	400

Source: Moreau, 2010

Table 5.45 Performance of MBR process (Nordkanal STP)

	COD	NH_4^+-N	NO_3^--N	PO_4^{3-}-P	TSS
Influent, mg/l	997-1,210	38.9	-	8.7	11,600- 14,800
Effluent, mg/l	17.4-22.1	0	2.57-3.5	0.26-0.3	-
Removal, %	98.2	100	-	96.8(A)	-

(A) Ferric chloride addition
Source: Moreau, 2010

c. Arakawa Sewage Treatment Plant, Japan

The operating condition and the performance of pilot scale MBR process in Japan (Applied for the recycled nitrification-denitrification process) are shown in Table 5.46 and Table 5.47 overleaf. Average flow of the MBR process is 54m³/h.

Table 5.46 Operating condition of MBR process (Arakawa STP)

HRT(hours)		SRT (days)	MLSS (mg/L)	Flux (L/m²/day)	Applied Vacuum (kPa)	Nitrified Recycle (% of influent)
Anoxic	Oxic					
3.0	3.0	20.0	9,200	500-1,000	15-35	300

Source: JSWA, 2003

Table 5-47 Performance of MBR process (Arakawa STP)

		pH	T-BOD	S-BOD	TSS	TN	TP	TColiform	Al
Inf. mg/L	(A)	7.15	191	43.1	222	34.1	4.75	3.6E+05	2.48
	(R)	6.97-7.47	92-497	21.5-78.6	65-867	21.5-58.7	2.74-9.51	9.6E+04 - 7.3E+05	0.78-11.63
Eff. mg/L	(A)	7.17	1.0	-	<0.4	5.0	0.52	0.24	0.03
	(R)	6.80-7.50	0.5-2.3	-	<0.4-0.7	2.1-6.9	0.09-2.16	ND-11.00	0.02-0.06
Removal (%)	(A)	-	99.5	-	99.8	85.3	89.1	99.9	98.8

T-BOD: Total BOD, s-BOD: Soluble BOD, TSS: Total suspended solids, T-N: Total nitrogen,

T-P: Total phosphorus, Al: Aluminium, T-coliform: Total coliform, ND: Non- detected, E: Exponent

Source: JSWA, 2003

d. MBR for park horticulture, Bangalore

The local authority has put up an automated MBR for horticulture of the city's famous Cubbon park. It uses the city sewage. It is under O&M by the contractor firm.

e. Porlock Sewage Treatment Plant, the United Kingdom

Porlock sewage treatment plant is the first sewage treatment plant introducing MBR process in the globe. The plant started operation in 1997 with capacity of 1.9 MLD.

The performance of the plant and number of membrane panels replaced are shown in Table 5-48 and Table 5-49 overleaf.

Table 5-48 Performance of MBR process

Parameter	BOD	COD	TSS
Influent(mg/l)	208	424	210
Effluent(mg/l)	<4	22	<2

Source: Churchhouse, 2003

Table 5.49 Number of membrane panels replaced

Period of membrane panels used (year)	Number of Membrane Panels		
	Installed (a)	replaced (b)	Ratio of b) / (a)
1	85,000	162	0.2
2	73,936	227	0.3
3	36,036	514	1.5
4	15,386	29	0.2
5	15,386	16	0.1
6	4,286	20	0.5
7	686	≤15	2.9

Source: JSWA,2003

f. Fukuzaki Sewage Treatment Plant, Japan

Fukuzaki sewage treatment plant is the first MBR plant in Japan.

The plant started operation in 2005 with capacity of 4.2 MLD. The plant also attempts to remove nitrogen and phosphorus.

The operating conditions and the performance of the MBR process are shown in Table 5.50 and Table 5.51.

Table 5.50 Operating condition of MBR process

MLSS, (mg/l)	Flux, (l/m^2/day)
8,000-12,000	400-600

Source: JSWA

Table 5.51 Performance of MBR process

	BOD		COD		T-N			T-P	
	Inf (mg/l)	Eff (mg/l)	Inf (mg/l)	Eff (mg/l)	Inf (mg/l)	Eff (mg/l)	Removal ratio (%)	Inf (mg/l)	Eff (mg/l)
FY2005	124	0.7	75.9	5.6	33.5	5.8	82.7	3.28	0.36
FY2006	159	0.9	85.2	5.4	38.3	4.9	87.2	3.61	0.20
FY2007	177	2.6	96.3	5.9	40.1	7.1	82.3	4.08	0.21
FY2008	177	3.8	109	6.0	46.1	8.3	82.0	4.66	0.08
FY2009	244	1.2	112	5.7	34.6	7.3	78.9	4.23	0.26
FY2010	263	1.5	98.4	5.5	33.1	7.3	78.0	3.49	0.29

Source: JSWA

5.18.11.3 Advantages and Disadvantages

The advantages and disadvantages of this process are as follows:

a. Advantages

  i. This process does not need primary and final sedimentation tanks, and disinfection facilities; therefore, it requires smaller space than conventional biological systems (generally around 1/3 of ASP system).

  ii. Since high MLSS concentration can be maintained in a reaction tank, MBRs operate at higher volumetric loading rates which result in lower hydraulic retention times. The low retention times mean that less space is required compared to a conventional system.

  iii. High MLSS concentration in a reaction tank enables dewatering of the excess sludge withdrawn from the reaction tank directly without thickening.

  iv. The effluent from MBRs is transparent containing almost no TSS. Organic matters (BOD) are well removed because of lower concentration of TSS compared with ASP process. Phosphorus can also be removed by adding coagulant in reaction tank.

  v. Long solids residence times (SRTs) of MBR, because of high MLSS concentration, is prone to nitrification reaction in its process. Therefore, if anoxic zone is applied in reaction tank, nitrogen is expected to be removed by biological nitrification and de-nitrification reaction.

  vi. E-coli is almost certainly blocked by MF membrane with pore size of less than 0.4 micro meters which is generally used for MBR system.

  vii. Treated sewage by MBR can be reused for various purposes such as toilet flushing, gardening, etc. without additional treatment.

viii. O & M works are easy and free from control of bulking and sludge recirculation because final sedimentation tank is not required. Monitoring and control of treatment process can easily be automated.

b. Disadvantages

i. Oil and grease is to be fully removed as otherwise membranes will get be choked and become unusable.

ii. Needs higher capital and operating costs than conventional systems for the same throughput.

iii. Needs a flow equalization tank to regulate fluctuation of the influent flows.

iv. Needs fine screens for pre-treatment to protect membranes.

5.18.11.4 Typical Design Parameters

According to the performance of the actual MBR processes and references on the MBR process, typical design parameters of this process and effluent quality are integrated as shown in Table 5.52 and Table 5.53.

Table 5.52 Typical design parameters of MBR process

COD Loading (kg/m^3/day)	F/M as MLVSS	SRT (days)	MLSS (mg/l)	Flux (l/m^2/day)	Applied Vacuum (kPa)	DO (mg/l)
1.2-3.2	0.1-0.4	5-20	5,000-20,000	600-1,100	4-35	0.5-1.0

Source: Metcalf & Eddy, 2003

Table 5.53 Typical effluent quality

Effluent				
BOD, mg/l	COD, mg/l	NH$_4^+$-N, mg/l	T- N, mg/l	Turbidity
<5	<30	<1	<10	<1

Source: Metcalf & Eddy, 2003

Important considerations related to design and maintenance of this process are as follows:

a. Since constant volume of influent needs to be supplied to membrane modules, it is necessary to install a flow equalization tank to regulate the fluctuation in influent rate of flow.

b. There are two basic configurations of membrane: hollow fibre bundles and plate membranes. There is no difference in the removal efficiency of these two configurations.

c. Since the permeation flux of membranes decreases as water temperature falls, capacity of membranes should be designed considering influent temperature.

d. The area of opening of division wall between anoxic and oxic tanks must be adequate so that reverse mixing between these tanks do not occur. Moreover, the reaction tank should be covered so that foreign objects do not enter it.

e. Total amount of air required in aeration tank should be designed considering amount of air required for biological treatment and for scrubbing the surface of the membranes.

f. It is recommended that the installation include one additional membrane tank/unit beyond what the design would nominally call for considering operation and maintenance.

g. Some types of membrane require a separate washing tank for membrane modules.

h. MBR systems are configured with the membranes actually immersed in the biological reactor or, in a separate vessel through which mixed liquor from the biological reactor is filtered.

5.18.11.5   Applicability

For new installations, the use of MBR systems allows for higher sewage flow or improved treatment performance in a smaller space than a conventional biological system using activated sludge because there are no installations of secondary sedimentation tanks, sand filters and disinfection facilities. This process has been used in the past only in smaller-flow systems due to the high capital cost of the equipment and high operation and maintenance (O&M) costs.

Presently, they are being increasingly used in larger systems. MBR systems are also well suited for some industrial and commercial applications. The high-quality effluent produced by MBRs makes it particularly suitable for reuse applications and for surface water discharge applications requiring extensive nutrient (nitrogen and phosphorus) removal.
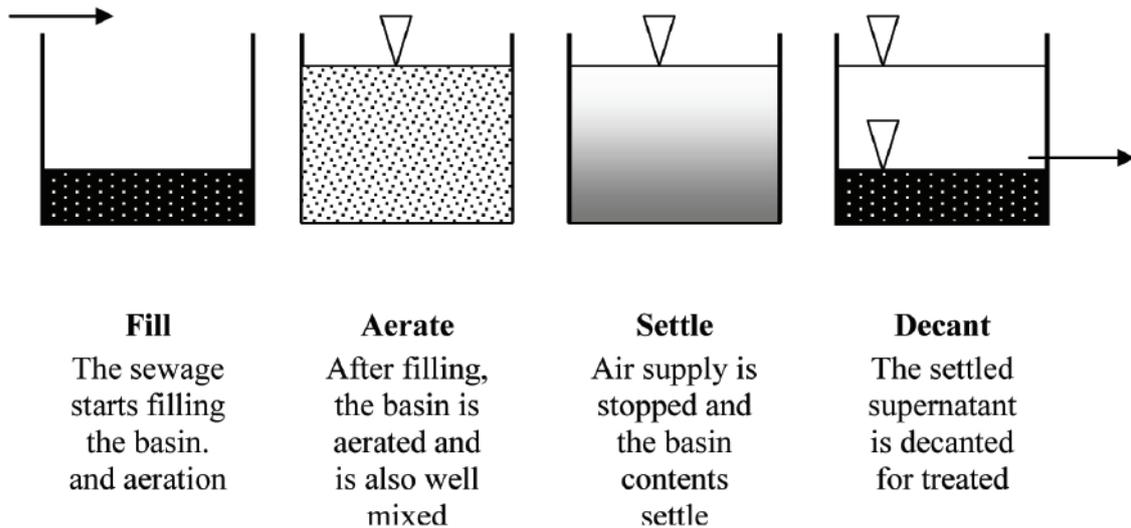
5.18.11.6   Guiding Principles

a. Maintenance of higher MLSS concentration is not easy. In practice, high MLSS concentration cannot be sustained. So smaller reaction tank is provided; and effluent BOD is not as desired. Adequate safety factor has to be considered for reaction tank.

b. Manufacturer can estimate the value of MLSS for pilot equipment and then design the plant based on the estimated MLSS. Pilot equipment cost can be borne by the owner.

5.18.12   Sequencing Batch Reactors (SBR)

5.18.12.1   Description

In its functional process scheme, a Sequencing Batch Reactor (SBR) is the same as the activated sludge process. The only difference is in the activated sludge process, the sewage flows through a primary clarifier, an aeration tank and then through a secondary clarifier continuously whereas in the SBR, the aeration and settling are carried out in batch mode one after the other in the same tank.

An illustrative schematic diagram is in Figure 5.77 overleaf.

Fill	**Aerate**	**Settle**	**Decant**
The sewage starts filling the basin. and aeration	After filling, the basin is aerated and is also well mixed	Air supply is stopped and the basin contents settle	The settled supernatant is decanted for treated

Note: All the four steps shown above take place in the same tank one following the other on a time interval basis but are shown separately in this figure for easy understanding.

Figure 5.77 Illustrative Schematic of SBR operation in the same Basin

Primary clarifiers do not seem to be provided. Consequently, at least two SBR basins are needed in parallel so that when one is in aeration, the other can be in settling and decanting of the supernatant. In fact the activated process can be referred to as continuous flow reactor (CFR). For this reason, the footprint on like to like basis of this type of SBR will be higher. In the CFR the suspended solids in the settling tank are constantly under simultaneous influence of opposing upward hydraulics of the overflowing treated sewage and gravitational setting of the suspended solids. In the SBR, this is got over by batch settling.

In fact, the CFR can also be designed with the settling tank alone in parallel modules and in batch settling alternatively. The SBR does have some advantages and they are addressed herein.

SBRs are typically configured and operated as multiple parallel basins. It aims to provide process and equipment performance, and variously include an instrumental control system that regulates timed sequences for filling, reaction, settling and effluent decanting. All these are referred to as one cycle of process control operation. It is the time duration between successive decanting sequences during which the liquid level moves from a lower water depth (bottom water level) to its fill depth (top water level) and back to its lower water depth (bottom water level). This volume progression takes place in repetitive sequences that permit reactive filling to be followed by solids liquid separation. The operational and process controls are governed as flows

a. A batch reactor consisting of a single tank equipped with an inlet for raw sewage, air diffusers, with associated compressors and piping for aeration; a sludge draw-off mechanism for waste sludge; a decant mechanism to remove the supernatant after settling; and a control mechanism to time and sequence the processes.

b. Decanting of the settled supernatant is carried out by equipment called as decanters. These consist of sharp edged weir plates over which the settled supernatant overflows similar to conventional clarifier weirs.

The scum baffles are provided before these weir plates similar to the primary clarifiers. The difference between clarifiers and these decanters is that in the case of clarifiers, the water surface remains constant and the weir plates are fixed permanently at that water surface. In the case of SBRs, the water surface will keep going down as the settled sewage is withdrawn because there is no inflow during this period. Hence, the weir plate has to move simultaneously down with the water surface and the collected settled sewage has to be discharged out of the SBR basin through a fixed pipe outlet. This is achieved by unique mechanisms called decanters. There are mainly three types of decanters namely (a) mechanized float controlled, (b) mechanized swing controlled and (c) hydraulically float controlled. These are shown in Figure 5.78 overleaf. The country has very limited experience on the performance of the various type of decanters. While selecting a decanter the competent authority may decide the type of decanter after ascertaining their field performance in the country or elsewhere in the world under similar conditions.

c. Wasting of surplus sludge typically occurs during the non mixed (aerated) stage. The sequence to take advantage of the higher concentrations of settled mixed liquor; wasting can equally take place in an aerated mixed condition.

d. SBR plants consist of a minimum of two reactors in a plant. When one reactor is in the fill and aeration mode, the other reactor can be in settling and decanting mode of the cycle.

e. In the reaction stage, the oxygen is supplied to the system within the time frame of the reaction cycle.

f. Each single SBR basin has the same floor area for all sequences in each cycle of operation.

As with CFRs, there are a number of types of SBRs all of which are easily differentiated. The main differences relate to their cyclic sequencing operation. The SBR efficiency derives from a capacity to maintain good sludge settling through batch settling. As with CFRs, nitrogen removal by biological nitrification-denitrification as also biological phosphorous removal by upstream anaerobiasis can also be built into the SBRs. Generally, the SBRs are reported in F/M ratios bordering on the extended aeration mode for the full quantity of the treated sewage.

However, these can also be used with primary settling and F/M ratios like in conventional ASP in CFRs to generate biomethanation from primary and excess volatile sludges and electricity production from the methane and thus save on electricity costs. There are variants of this basic SBR technology. Some of these are as follows:

a. Intermittent Feed and Intermittent Decant SBR Process

In this process, the inflow and outflow are intermittent at the beginning and end of the treatment cycle. Interrupted inflow during the settle and decant sequencing provides the best possible environment for solids liquid separation. Operation with specific initial fill only sequencing to generate 30% to 50% bulk in basin biological selectivity mechanisms against filamentous sludge bulking; typically this is thirty percent to fifty percent of the fill-react sequence in a cycle.

A mechanized cum float arrangement. It is stated as a circular weir with a surrounding floatation device also acting as a scum baffle. The decanting rate is stated as controllable by the weir elevation by operating the three screw jacks with a single motor and depth adjustment of the weir by electric actuators and that the float is raised above the sewage during aeration and is rested on tripod supports for maintenance. Source: www.as-h.com

A hydraulic float arrangement. It is stated as using a draw tube with solid excluding plugs and kept buoyant supported by a FRP foam filled float parallel to the tube and both these integrally spanning the width of the basin at one shorter side and in multiple such decanters as needed and rested on supports erected in construction within the basin. The decant valve opening is stated to control the rate of decanting. Source: www.water.siemens.com

The mechanized swing decanter. It is stated as a motor controlled mechanism which holds the decanting weir and scum baffle and swings it down to the desired rate of decanting and adjustable at the motor and the rate of decent of the weir. Source: www.alibaba.com


Float type decanter. The decanting pipe is stated to have floating pontoons and evenly distributed decanting holes to fitted with the company's patented closing function, preventing the sludge from entering the decanting pipe holes..

Illustrations are only for familiarity of explanations & not standalone endorsements

Figure 5.78 Types of Decanters

Cyclic flow interruption during settle and decant sequences to positively prevent by pass flow of untreated sewage that can otherwise degrade effluent quality; all influent sewage receives aeration during a cycle. Operational protocols that maximize nitrogen removal through conventional sequenced aeration for nitrification followed by sequenced anoxic mixing for denitrification for which total cycle times are typically 6 hours. In addition appreciable denitrification also takes place during settling (Kazmi and Furumai, 2000).

This conventional SBR configuration uses sequences described as Fill, React, Settle, Decant, and Idle are shown in Figure 5.79.
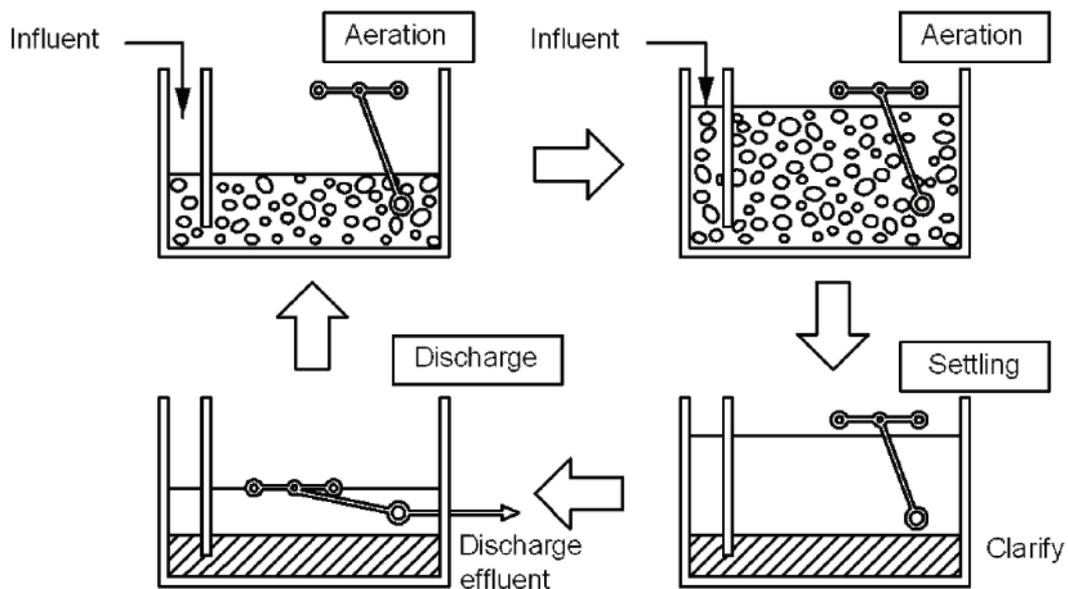
Source:Nishihara Environment Co., Ltd.

Figure 5.79 Typical operating cycles of intermittent SBR process

c. Continuous Flow SBR Process

It is stated that the need for at least two parallel modules and hence relatively larger foot print than CFR, has been sought to be got over in this process where a single reactor is divided as the pre-treat zone and downstream main-react zone and interconnected at floor level.

To start with, the liquid depth in both these zones is reduced by the depth by which the supernatant is decanted in the main-react zone. At this stage, the raw sewage continues to enter the pre-treat zone reacts with the returned MLSS and depending on the objectives, this can be run as anaerobic, anoxic or aeration or a sequential mix. The reacted MLSS flows into the main-react zone at the floor level and at an appropriate time, the air agitation is cut off permitting the solids liquid separation and the supernatant is drawn off by the decanter of the swing down or float type and sludge wasting takes place simultaneously. An advantage stated is the elimination of a separate return sludge pumpset.

An illustrative schematic is shown in Figure 5.80 overleaf.
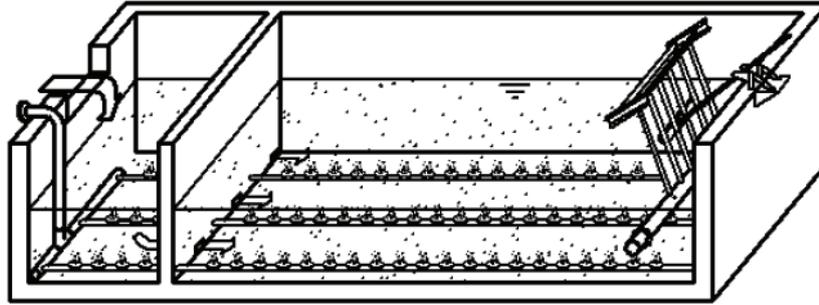
Source:http://www.ittwww.com.au/brochures/abj_brochures.pdf

Figure 5.80 Typical configuration of Continuous Flow SBR

d. Selector Zone Incorporated SBR Process

Bioreactors operating in low F/M ratio and long sludge ages may sometimes experience problems due to filamentous organisms causing bulking sludge.

A customary solution is to provide a selector tank with a hydraulic retention time of about 30 minutes before the bioreactor and operate at a high F/M ratio which ensures growth of floc forming microorganisms while suppressing filamentous growth. The selector also helps in removing some of the organic phosphorus without chemical addition. This is also used in the case of SBR. The reactor is partitioned initially for the selector zone.

e. Chemically Enhanced SBR Phosphorous Removal Process

Addition of metal salts if necessary by supplementing the bicarbonate alkalinity is also another customary solution in removing phosphorous which remains in excess of the uptake.

This is also used in the case of SBR by adding the alkalinity as needed in the react cum fill stage and the metal salt in the subsequent react stage.

All sewage received in a cycle is similarly treated as in the conventional SBR. Purposely designed and controlled reaction environments of aerobic, anoxic and anaerobic are a feature of this variation of a conventional SBR.

5.18.12.2 Application Examples

a. Mundhwa Sewage Treatment Plant, Pune, India

The design details of the Mundhwa STP reproduced in Table 5.54 (overleaf) are extracted from the IIT Roorkee report August 2010.

The performance results are extracted from the IWWA 2011, It is stated that the raw sewage values of BOD, SS, TKN and PO4-P are 205 mg/l, 262 mg/l, 45 mg/l and 2.3 mg/l and the treated sewage values of BOD, SS, TKN are reduced to less than 10 mg/l, and PO4-P is reduced to 0.7 mg/l.

Table 5.54 Design criteria stated as used in the 45 MLD Cyclic Activated Sludge process based SBR at Mundhwa STP in Pune

Design Flow • Average flow • Peak factor • Peak flow	45 MLD 2.25 times average flow 95 MLD
Influent Quality to SBR Basin • BOD • SS • TKN • TP • Faecal coliform	250 mg/l 350 mg/l 45 mg/l 5 mg/l 10^6 MPN/100 ml
Effluent Quality of SBR Basin • BOD • SS • TKN • TP • Faecal coliform	< 10 mg/l < 10 mg/l < 10 mg/l < 2 mg/l < 100 MPN/100 ml
No. of basins	4
Each basin area	1431 m^2
Basin foot print	127.5 m^2/1000 m^3/d
Decant volume	3377 m^3
Decanter loading	300 m^3/1000 m^3/d
Hourly flow rate	1875 m^3/h
Hourly flow rate to each basin	937 m^3/h
No. of cycles per day/basin • Filling and aeration • Settling phase • Decanting phase Total cycle time	8 90 min. 45 min. 45 min. 3 h
Hours of aeration time/day/basin	12 h
MLSS MLVSS F/M	4300 mg/l 3440 mg/l 0.08
HRT	16.30 h
SRT	14.86 days
Selector (Anoxic) Zone • Number of selector compartments/basin • Retention time in selector zone	1 50 min.
Disinfection • Chlorine dose after SBR Basin	2.5 mg/l

Source: IIT Roorkee, August 2010

b. Culver Sewage Treatment Plant, USA

The operating conditions and the performance of the Culver SBR process in USA are shown in Table 5.55 and Table 5.56.

Table 5.55 Operating conditions of the Culver SBR process

| Average flow (MLD) | Mode of Operation (minutes) | | | | | | | Number of Basins | MLSS (mg/L) | F/M (a) | SRT (days) |
	Fill and Mix	Fill and Aeration	React	Settle	Decant	Idle	Total cycle time				
1.34	54	126	42	42	42	60	366	2	5,000	0.08-0.16	15.5

Source: USEPA, 1986, USEPA, 1983

Table 5.56 Performance of the Culver SBR process

	BOD	TSS	NH_4^+-N + NO_3^--N	T - P
Influent (mg/L)	170	150	22.0	6.5
Effluent (mg/L)	10.5	5.5	2.4	0.75
% Removal	94	96	89	88

Source: USEPA, 1986, USEPA, 1983

5.18.12.3   Advantages and Disadvantages

a. Advantages

  i. One single reactor basin provides all of the unit operations and processes that require two separate basins in a conventional activated sludge plant configuration that can provide an effluent quality suitable for reuse. Equalization, primary clarification (in most cases), biological treatment, and secondary clarification can be achieved in a single reactor vessel.

  ii. This process can be operated and controlled with flexibility for efficient removal of organic matter, suspended solids, nitrogen, and phosphorus under all loading conditions. Provides enhanced organic phosphorus removal with or without chemical augmentation.

  iii. This process can control the growth of filamentous bacteria and hence prevent bulking of activated sludge.

  iv. This process saves capital cost by eliminating final sedimentation tanks. As secondary sedimentation tanks are not required in this process, footprint area needed is also minimal as simultaneous multiprocessing takes place in a single reactor basin (approximately 100m²/1000m³ only needed for SBR Tanks).

v. Can be used with primary clarifiers and power generation configurations where the ratio of VSS:TSS is high.

vi. Allows for easy modular expansion for population growth, modular configurations and cyclic operation is easily managed to provide continuous inflow and outflow hydraulic profiles, dispensing with the need for outflow hydraulic balancing

b. Disadvantages

i. Compared to the conventional activated sludge system, a higher level of sophistication and maintenance can be associated with more automated switches and valves.

ii. Basin depth should be sufficient to provide an adequate clear water depth over the sludge blanket to prevent settled solids entrainment.

iii. In small single stream SBR systems approximately less than 10 MLD, effluent flow balancing may be needed for downstream processing, such as filtration or disinfection.

iv. Short-circuiting of influent conservative parameters (ammonia nitrogen, orthophosphate) under the non-interrupted inflow protocol may be a process failure consideration in some SBRs.

v. Larger capacity aeration system, relative to aeration time per cycle and per day, is required compared as to conventional activated sludge system.

vi. The potential for discharging floating or settled sludge during the decant phase with certain SBR configurations.

vii. Potential plugging of aeration devices during selected operating cycles depending on the aeration system used by the manufacturer.

viii. There should be sufficient allowance of clear water depth from the sludge blanket to minimize sludge carryover. The volume of water decanted should be limited to prevent scouring of solids.

ix. All the SBR plants must be designed to cater to the peak flows. A minimum of two tank system is required.

5.18.12.4 Typical Design Parameters

A compilation of typical process details that would feature in the use of SBR facilities is mentioned in Table 5.57. It has to be recognized that as with all similar technologies, these are only of informative value for India and it is mandatory that there is a demonstrated available level of Indian expertise and support services for the design of SBR systems and its operational methodology for India which is to be hereafter evolved with reference to a validation of design vs. actual performance of SBRs built in India.

One of the classical difficulties that pertain to establishing the design parameters for SBR is the biomass metabolism. In the ASP and CFR, it takes place under "steady state conditions" where a steady BOD profile from inlet to outlet is existing in the aeration tank irrespective of time.

Table 5.57 Typical process parameters for SBR configurations (for unsettled sludge)

S. No.	Parameters	Units	Continuous Flow and Intermittent Decant	Intermittent Flow and Intermittent Decant
1	F/M ratio	d^{-1}	0.05 - 0.08	0.05 - 0.3
2	Sludge Age	d	15 - 20	4 - 20
3	Sludge Yield	kg dry solids/ kg BOD	0.75 - 0.85	0.75 - 1.0
4	MLSS	mg/L	3,000 - 4,000	3,500 – 5,000
5	Cycle Time	h	4 - 8	2.5 - 6
6	Settling Time	h	> 0.5	> 0.5
7	Decant Depth	m	1.5	2.5
8	Fill Volume Base	-	Peak Flow	Peak Flow
9	Process Oxygen			
	BOD	kg O_2/kg BOD	1.1	1.1
	TKN	kg O_2/kg TN	4.6	4.6

* For Phosphorous ≤ 1 mg/L, after bio-P removal, metal precipitant (Fe^{3+} or Al^{3+}) shall be added. Sludge yield factor and sludge age not applicable for primary settled sewage; typical primary TSS removal 60%, BOD 30%.

In the case of SBR, the biomass metabolism takes place under "unsteady state conditions" where the BOD profile decreases with time during the batch time interval. Thus, calculating the oxygen requirements is a challenge depending on many factors. In actual practice, the oxygen requirement is calculated as though it is a steady state condition as in the CFR and then the rate of air delivery to the basin is calculated by delivering the entire volume of air in the actual aeration interval. This will need a much bigger air compressor and air diffuser system.

However the motor is controlled by VFD and thereby the delivery of the compressor is gradually adjusted down to suit the real need or maintained as it is based on maintaining the required residual D O in the basin at the end of aeration. It is a unique feature in the design of SBR aeration facilities.

However, it needs a focussed study to establish the actual oxygen uptake rate as a function of aeration interval to make future designs more realistic. Such data as validated by actual observations is not found in literature.

5.18.12.4.1   Disinfection tank volume

The capacity of the chlorine contact tank for the treated sewage of batch processes such as the SBR will be based on 30 minutes detention time of the rate of decant flow calculated as volume of decant flow divided by the duration of decanting of any one or multiple reactors decanting simultaneously.

5.18.12.5   Applicability

Municipal sewage is successfully treated in SBR systems. As with conventional activated sludge plants, SBRs can be used for all plant sizes. Current practice examples large scale facilities in municipal applications to about 270 MLD, (Goronszy, 2008), Especially where land availability is limited; plants can easily be installed on a multi-level basis, like the one reported to be in use at Thailand as in Figure 5.81.

Figure 5.81 Yannawa STP, Thailand

5.18.12.6   Guiding Principles

With proper anticipations in the design stage, the SBR process can be installed with good flexibility to adapt to future regulatory changes for effluent parameters such as for nutrient removal. As with conventional activated sludge variants, there are several SBR variants each of which requires their own design considerations. Design and operation for efficient nitrogen removal provides enhanced process stability, especially with operating temperatures greater than 20ºC. It is necessary to bear in mind that there are no procedures to design the SBR, like all other biological processes for obtaining a desired removal of coliform organisms. At best one can cite another functioning SBR but the result of that STP need not necessarily be the same for the new SBR even though it is designed on the same lines. It applies to almost all such biological processes. Hence, provision for variable chlorine dosage of the SBR basin effluent should be made.

5.18.13   Moving Bed Biofilm Reactors (MBBR)

5.18.13.1   Introduction

The moving bed biofilm reactor (MBBR) is based on the biofilm carrier elements. Several types of synthetic biofilm carrier elements have been developed. These biofilm carrier elements are floated in the mixed liquor in the aeration tank and are kept floating by the air from the diffusers. They have a tendency to accumulate at the top zones. Hence wall mounted mixers propel the media downwards so that they again float and are in circulation in the mixed liquor. They are retained by suitably sized sieves at the outlet.

This process is intended to enhance the activated sludge process by providing a greater biomass concentration in the aeration tank and thus offer the potential to reduce the basin volume requirements. They have also been used to improve the volumetric nitrification rates and to accomplish the denitrification in aeration tanks by having anoxic zones within the biofilm depth. Because of the complexity of the process and issues related to understanding the biofilm area and activity, the processes design is empirical. There are now more than 10 different variations of the processes in which a biofilm carrier material of various types are suspended in the aeration tank of the activated sludge process. There are many examples of such activated sludge treatment process with suspended biofilm carrier in the world. In this section, some of the more widely cited processes such as the Captor®, Linpor®, Pegasus®, and Kaldnes® are described and some design considerations and parameters are cited.

5.18.13.2   Description

a. Captor® and Linpor® process

In the Captor® and Linpor® processes, foam pads with a specific density of about 0.95 g/cm^3 are placed in the bioreactor in a free-floating fashion and retained by an effluent screen. The pad volume can account for 20% to 30% of the reactor volume. Dimensions of the carrier materials are presented in Table 5.58.

Table 5.58 Dimensions of the carrier materials

Name	Carrier specifications	
	Material	Dimension, mm
Captor®	Polyurethane	30 x 25 x 25
Linpor®	Polyurethane	10 to 13 cubes

Source: Metcalf & Eddy, 2003

Mixing from the diffused aeration circulates the foam pads in the system, but without additional mixing methods. They may tend to accumulate at the effluent end of the aeration basin and float at the surface. An air knife has been installed to clean the screen continuously and a pump is used to return the carrier material to the influent end of the reactor. Solids are removed by a conventional secondary clarifier and wasting is from the return line as in the ASP.

The principal advantage for the sponge carrier system is the ability to increase the loading of an existing plant without increasing the solids load on existing secondary clarifiers, as most of the biomass is retained in the aeration basin. Loading rates for BOD of 1.5 to 4.0 kg/m^3/d with the equivalent MLSS concentration of 5,000 to 9,000mg/L have been achieved with these processes. Based on the results with full-scale and pilot-scale tests with the sponge carrier installed, it appears that the nitrification can occur at the apparent lower SRT values, based on the suspended growth mixed liquor, than those for activated sludge without internal carrier. Compared to other biofilm carriers, Captor® and Linpor® Cubes are larger; meaning the openings of the screen used to block the outflow of the biofilm carriers can also be larger.

Therefore, there is less clogging of the screen due to scum. Furthermore, Linpor® Cubes are stated to have better durability and it is stated that even after a long period use, replacement is not necessary. The Linpor® system was developed by Linde AG in the mid-1970's, Linpor®-CN Process and Linpor® Cubes carrying sludge are shown in Figure 5.82 and Figure 5.83.
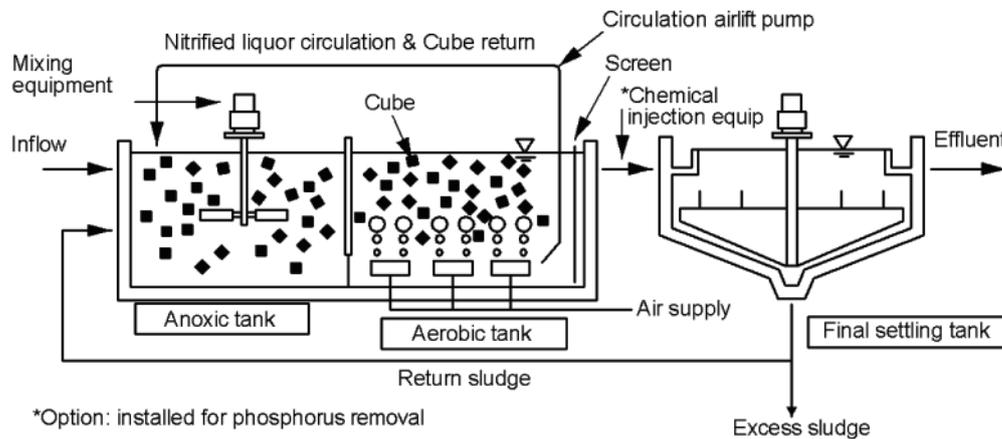
Source: Nishihara Environment Co., Ltd.
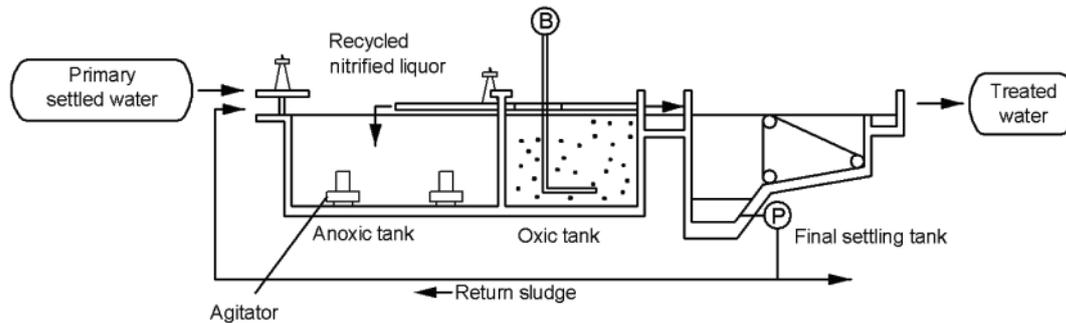
Figure 5.82 Linpor®-CN process flow diagram

Source:Nishihara Environment Co., Ltd.

Figure 5.83 Linpor® Cubes carrying sludge (12×12×15 mm)

b. Pegasus®/Bio-cube process

The Pegasus system is based on the immobilization of nitrifying bacteria in organic gel pellets called bio-n-cubes which are non-biodegradable organic matrix consisting of a mixture of polyethylene glycol and nitrifying activated sludge. The number of immobilised autotrophic bacteria is independent of the sludge age and thus higher than in comparable low-loaded activated sludge systems. The immobilization prevents the autotrophic bacteria from wash-out, improves the nitrification kinetics and results in a lower temperature dependency for the ammonia removal process. The bio-n-cubes are maintained in suspension in the aeration tank and wash-out is prevented by a retention grid at the outlet of the tank.

If the existing volume of the activated sludge tank enables the hydraulic retention time of the sewage to be at least five hours, total nitrogen removal is stated as implemented directly in the activated sludge by means of the Pegasus immobilised culture system. The biopellets used in the Pegasus process, called bio-n-cubes are relatively small in size and produced in a way to ensure a balance between oxygen transfer, biomass growth and suspendability. The annual wear rate of volume of bio-n-cubes is stated as 1%. The bio-n-cubes are proprietary products of Hitachi Plant Technologies, Ltd. The schematic is shown in Figure 5.84. The bio-cubes are shown in Figure 5.85. The parameters of the pellets used are given in Table 5.59.

Source:Hitachi Plant Technologies, Ltd., 2010

Figure 5.84 Pegasus® process flow diagram
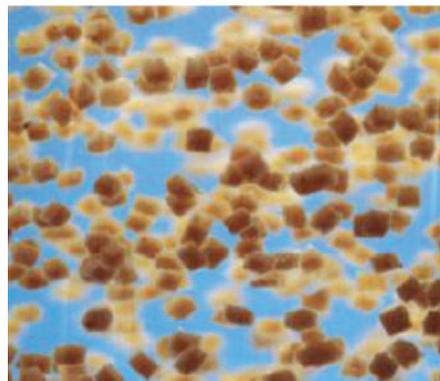
Source: Hitachi Plant Technologies, Ltd 2010

Figure 5.85 Pegasus® bio-n-cubes (3×3×3 mm)

Table 5.59 Parameters of the pellets of the Pegasus process

Pellet material	Polyethylene glycol (PEG)
Pellet size	3×3×3 mm
PEG volume fraction	10 - 20%
Micro-organism fraction	2%
Density	1.03 g/cm^3
Surface area	700 m^2/m^3 reactor volume
Biofilm thickness	~ 60 μm

Source: Hitachi Plant Technologies, Ltd., 2010

c. Kaldnes® process

This technology was developed by the Norwegian firm of M/S Kaldnes Miljoteknologi. The processes consist of adding small cylindrical shaped polyethylene carrier elements (specific gravity of 0.96 g/m^3) in aerated or non-aerated tanks to support biofilm growth. The small cylinders are about 10 mm in diameter and 7 mm in height with a cross inside the cylinder and longitudinal fins on the outside. The biofilm carriers are maintained in the reactor by a perforated plate (5×25 mm slots) at the tank outlet. Air agitator or mixers are used to circulate the packing continuously. The packing may fill 25% to 50% of the tank volume. The specific surface area of the packing is about 500 m^2/m^3 of bulk packing volume. The typical reactors are shown in Figure 5.86. The biofilm carriers are shown in Figure 5.87. The schematic is shown in Figure 5-88 overleaf.
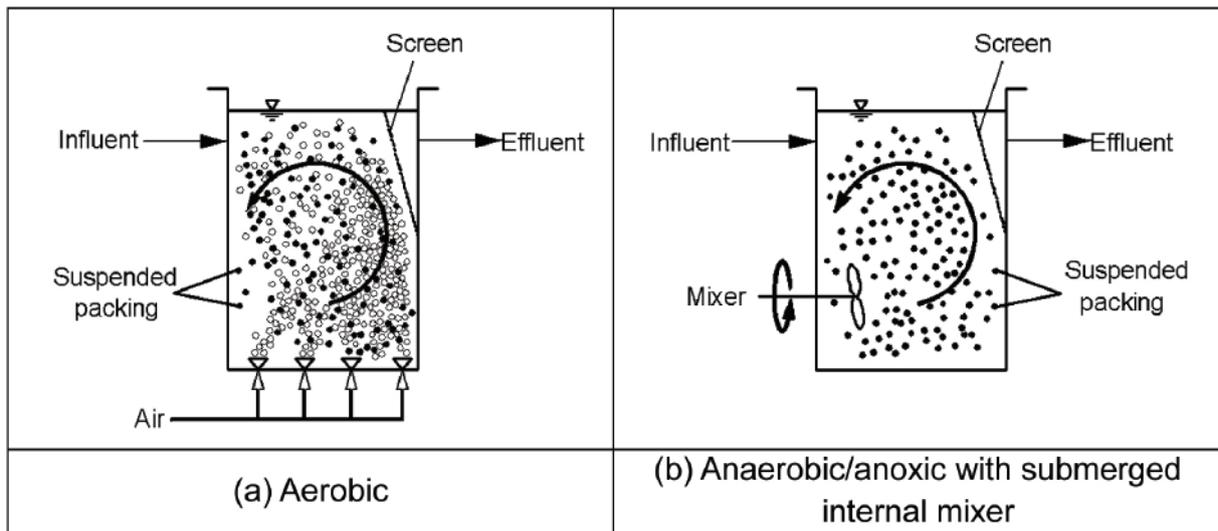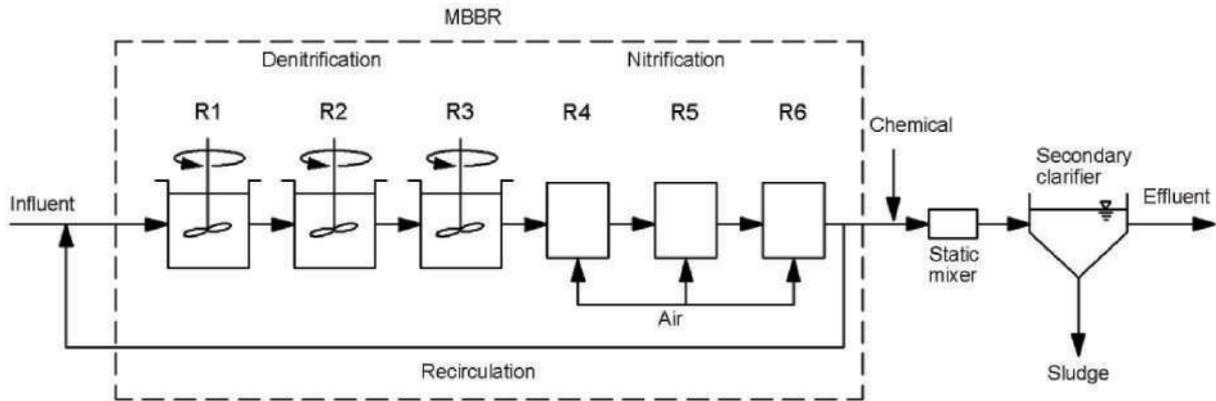

Source: Metcalf & Eddy, 2003

Figure 5.86 Typical reactors of MBBR process

Source:Stowa Webpage

Figure 5.87 Kaldnes biofilm carriers (About 10 mm diameter × 7 mm height)

Source: Metcalf & Eddy, 2003

Figure 5.88 Schematic flow diagram of MBBR process for removal of BOD and nutrients

d. FAB Technology

The fluidized aerobic bioreactor includes a tank in any shape filled up with small carrier elements. The elements are specially developed materials of controlled density such that they can be fluidized using an aeration device.

A biofilm develops on the elements, which move along with the effluent in the reactor. The movement within the reactor is generated by providing aeration with the help of diffusers placed at the bottom of the reactor. The thin biofilm on the elements enables the bacteria to act upon the biodegradable matter in the effluent and reduce BOD/COD content in the presence of oxygen from the air used for fluidization. The technology is shown in Figure 5.89.
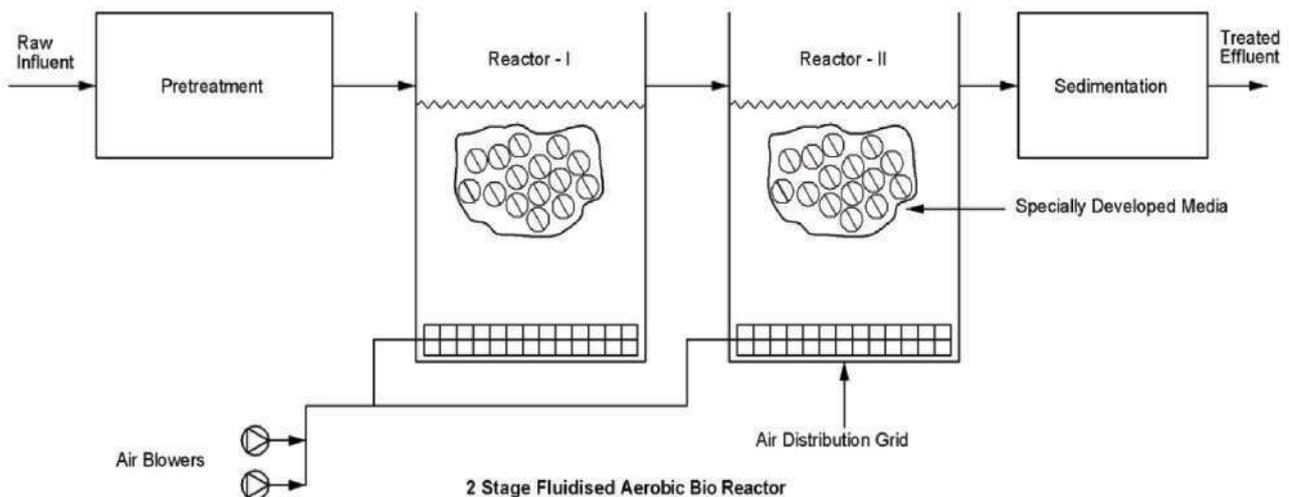

Figure 5.89 FAB technology

The merits of the FAB technology as stated by its promoter is mentioned in Table 5.60 overleaf.

Table 5.60 Merits of FAB technology as stated by its Promoter

Features	Benefits
Attached growth process	No sludge recycle No monitoring of MLSS Low sludge production
High Biofilm surface area	High loading rates Compact plants Small foot print
Fluidised Bed	Non clogging design Better oxygen transfer efficiency Reduced power consumption Reduces coliform Low maintenance Tank of any shape can be utilized

5.18.13.3   Application Examples

a. Captor® and Linpor® process

The design details and the performance data of the Linpor® CN Process for a biological nitrification-denitrification STP of Freising, Germany are mentioned in Table 5.61 and Table 5.62.

Table 5.61 Design Details of the biological nitrification-denitrification Linpor process
(Freising, Germany)

	Flow rate (MLD)	MLSS(mg/L)			Carrier Volume (%)	F/M (1/d)
		Suspended	Fixed	Average		
Design	23.6	3,800	15,000	5,800	22	0.12
Operation	21.4	5,700	18,000	7,700	20	0.04

Source: Gilligan and Morper, 1999

Table 5.62 Performance of the biological nitrification-denitrification Linpor process
(Freising, Germany)

	Design		Operation	
	Influent	Effluent	Influent	Effluent
BOD (mg/l)	222	-	106	3
COD (mg/l)	397	-	208	-
TKN (mg/l)	46.6	-	29	<0.6
NH_4^--N (mg/l)	35.9	<5	16	<0.1
NO_x^--N (mg/l)	2.6	-	-	9.9
Total-N (mg/l)	49.2	<18	29	9.9

Source: Gilligan and Morper, 1999

b.   Pegasus®/Bio-cube process

The design details and the performance data of Pegasus process for a biological nitrification-denitrification STP of Munakata City, Japan are in Table 5.63 and Table 5.64.

Table 5.63 Design details of the biological nitrification-denitrification Pegasus process
(Munakata City, Japan)

	Maximum daily low (m3/day)	11,300
Primary settling tank	Surface-loading ($m^3/m^2/day$)	36
	Retention time (hours)	1.7
Anoxic tank	Capacity (m^3)	2,008
	Retention time (hours)	4.3
Oxic tank	Capacity (m^3)	1,436
	Retention time (hours)	3.0
	Pellet dosing ratio (%)	7.5
	Circulation ratio of nitrified liquor to influent flow (-)	1.5-3.0
Final settling tank	Surface-loading ($m^3/m^2/day$)	25
	Retention time (hours)	2.9
PAC feeder	Dosage of coagulant ($L/1,000m^3$)	70
	A/p (-)	1

Source: Hitachi Plant Technologies, Ltd.

Table 5.64 Performance of the biological nitrification-denitrification Pegasus process
(Munakata City, Japan)

Water quality	Influent	Effluent from secondary settling tank	After rapid filtration
BOD (mg/l)	210	14.2	10
SS (mg/l)	250	20.5	5
T-N (mg/l)	40	10.2	10
T-P (mg/l)	6	0.4	0.3

Source: Hitachi Plant Technologies, Ltd.

c. Kaldnes® process

The performance data and the schematic flow diagram of the Kaldnes process in Lillehammer STP of Norway, which applies the biological nitrification-denitrification and phosphorus removal by chemical precipitation, are shown in Table 5.65 and Figure 5.90.

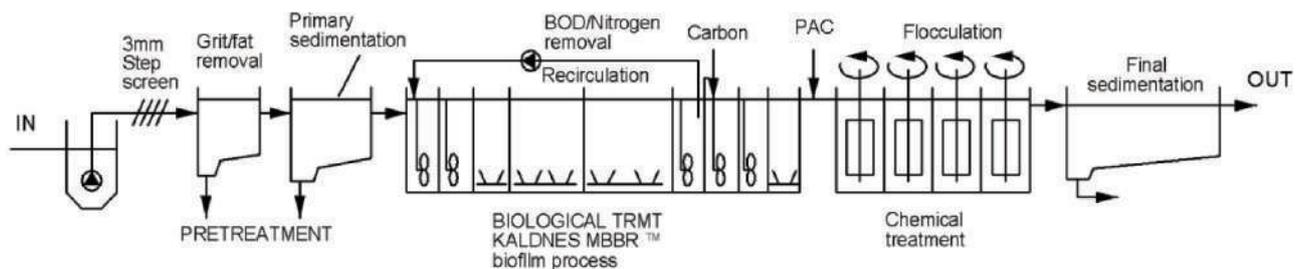Table 5.65 Performance of the Lillehammer Kaldnes process

	Influent (mg/l)	Effluent (mg/l)	Removal (%)
BOD_7	81.67	3.33	96
T-N	25.46	5.17	80
T-P	3.67	0.08	98

STP for Nitrogen and Phosphorus removal Lillehammer, Norway.
Dimensioning flow: 1,200 m³/hr, Maximum flow: 1,900 m³/hr
Design Temperature: 10ºC
Source: Kaldnes Web page

Source: Kaldnes Web page

Figure 5.90 Schematic flow diagram of Lillehammer Kaldnes process for removal of BOD and nutrients

5.18.13.4   Advantages and Disadvantages

The advantages and disadvantages of these MBBR processes are as follows:

a. Advantages

i. These processes can enhance the activated sludge process by providing a greater biomass concentration in the aeration tank and thus offer the potential to reduce the basin size requirements.

ii. Packing materials can maintain the concentration of nitrifying bacteria in the aerobic tank at a high level, and the nitrification reaction proceeds efficiently. Hence, these processes can improve the volumetric nitrification rates and accomplish the denitrification in aeration tanks by having anoxic zones within the biofilm depth.

iii. These processes can be used for the upgrading of existing STP, especially when space is an issue.

iv. These processes can upgrade the plant by reducing the solids loading on the existing sedimentation tank.

b. Disadvantages

i. Because of the complexity of the process and issues related to understanding the biofilm area and activity, the processes designs are empirical and based on prior pilot-plant or limited full-scale results.

ii. When upgrading existing treatment plants that operate without primary settling and rather large screen sizes, the carrier material should be chosen appropriately to prevent clogging.

iii. In the aeration tank of these processes, the concentration of dissolved oxygen (D O) has to be relatively high because the D O concentration is the limiting factor in the biofilm processes. A high driving force in terms of D O concentration across the biofilm is therefore required.

5.18.13.5   Typical Design Parameters

a. Linpor® process

Typical design parameters of the Linpor process are shown below:

i.   MLSS suspended: 3,800 (mg/L)
ii.   MLSS fixed: 15,000 (mg/L)
iii. MLSS total: 5,800 (mg/L)
iv.   Carrier volume: 22 (%)
v.   F/M: 0.12 (1/d)
(Gilligan and Morper, 1999)

b. Pegasus®/Bio-cube process

Typical design parameters of the Pegasus process are shown below:

i.   HRT (Anoxic tank + Oxic tank): 6-8 (hours)
ii.   Carrier volume: 10-20 (%)
iii. Circulation ratio of nitrified liquor to influent flow: 1.5-3.0 (-)
iv.   Final sedimentation tank hydraulic application rate: about 1.0 (m/hr)
(Hitachi Plant Technologies, Ltd.)

c. Kaldnes® process

Typical design parameters of the Kaldnes process are shown below:

i.   Anoxic detention time: 1.0-1.2 (hours)
ii.   Aerobic detention time: 3.5-4.5 (hours)
iii. Biofilm area: 200-250 (m^2/m^3)
iv.   BOD loading: 1.0-1.4 (kg/ m^3/d)
v.   Final sedimentation tank hydraulic application rate: 0.5-0.8 (m/hr)
(Metcalf & Eddy, 2003)

5.18.13.6   Applicability

The MBBR processes are used in municipal sewage treatment for BOD removal and nutrients removal. These processes are frequently used for upgrading an existing plant, especially when space is an issue. The existing reaction tanks can be either retrofitted with Linpor®, Pegasus®, or Kaldnes® or similar other such technologies in the world. The one handicap is the inability to derive the design guidelines for an STP. This is because, depending on the media that is used the available area for microbes to grow in a given volume of the reaction tanks varies rather widely and this compounds the formulation of a fundamentally reliable design equation. At this moment, heavy reliance on the eventual builders of these MBBR type STPs and the realization as well as the intangibles of their guarantees seems unavoidable.

5.18.13.7   Guiding Principles

a. There are several types of MBBR processes and the design guidelines should be studied and evaluated consciously.

b. Life cycle of media of MBBRs is uncertain as of now and it appears that one possible method of sustaining competition may be to opt for a contract including the replacements for a specified number of years as part of the contract itself.

5.18.14   Fixed Bed Biofilm Activated Sludge Process

5.18.14.1   Description

The FBAS process is an essentially an activated sludge attached growth process where the plant roots provide the area for the biofilm to develop and grow. The aeration system is divided into a series of biological reactors where fixed biofilm is maintained in every stage of the process. Biodegradation of influent contaminants takes place mainly with the help of fixed biological cultures, where plant roots are used as biofilm carriers; additional textile media is used in the reactors as additional biofilm carriers. As a standard feature of the technology the reactors are covered by a shading structure or a greenhouse. As the influent travels through the cascade, the available nutrient quantity is consumed and as a result, the composition of the ecosystem fixed in the biofilm changes from reactor to reactor, gradually adapting itself to the decreasing nutrient concentration. In each cascade stage, a specially adapted ecosystem will form, thus maximizing the decomposition of contaminants. It is stated that 32 plants with such technologies have been set up in different countries including Hungary, China and France, etc. in last 10 years. However, it will be useful to demonstrate this project under Indian conditions.

(Organica^{TM} technology) is stated to enhance the forces of nature to purify the sewage by harnessing the metabolic processes of living organisms that digest organic pollutants. In addition to the bacteria found in traditional activated sludge systems, the Organica STP are stated to be inoculated with 3,000 species of plants, animals, and microbes. The STP consist of a series of aerated reactors, filtration units and final polishing units. Plants with extensive root systems are placed on a supporting mesh slightly below the liquid level in the open aerobic reactors.

The roots of these plants, suspended 1.5 metres into the water, provide a healthy habitat for the bacteria and a whole range of other organisms such as protozoa, zooplankton, worms, snails, clams and even fish. As sewage flows through the technology train, different ecosystems develop in each tank. It is stated that there are two types of Organica treatment process. One is the Organica Fed Batch Reactor (FBR) process which combines conventional Sequence Batch Reactors (SBR) and continuous flow sewage treatment technologies. The other is the Organica cascade which is a continuous flow treatment process through a series of connected biological reactors.

5.18.14.2 Application Examples

a. Organica FBR (Fed Batch Reactor) process

i. Shenzhen, China

- Technology applied: Single train FBR process

- Footprint: 975 m^2

- Hydraulic Capacity: 400 m^3/day

- Community Served: 1,700 people

ii. Etyek, Hungary

- Technology applied: Two train FBR process

- Footprint: 570 m^2

- Hydraulic Capacity: 1,100 m^3/day

- Community Served: 10,000 people

iii. Le Lude, France

- Technology applied: Two train FBR process

- Footprint: 380 m^2

- Hydraulic Capacity: 815 m^3/day

- Community Served: 6,000 people

iv. Telki, Hungary

- Technology applied: Two train FBR process

- Footprint: 360 m^2

- Hydraulic Capacity: 800 m^3/day

- Community Served: 8,000 people

b.   Organica cascade process

i. Budapest Hungary

- Technology applied: Cascade process

- Footprint: 340 m^2

- Hydraulic Capacity: 280 m^3/day

- Community Served: 6,000 people

ii. Szarvas, Hungary

- Technology applied: Cascade process

- Footprint: 540 m^2

- Hydraulic Capacity: 1,600 m^3/day

- Community Served: 10,000 people

Independent evaluation of the results of the performance is not readily traceable in literature.

5.18.14.3   Advantages and Disadvantages

The advantages and disadvantages of this process are as follows:

a. Advantages

i. The process is stated to require much lesser land area than conventional activated sludge

ii. The process is stated to be odourless and hence the STPs are stated to be easily built in urban area with no negative impact on the value of adjoining areas.

iii. It is stated that it can operate at a much lesser loading rates during initial days of setting up the plant in new habitations and it is stated that due to small area requirements, this technology can offer decentralized solutions and recycling water in local areas.

iv. The technology allows for design flexibility and can be adopted for nutrient removal such as phosphorous and nitrogen, which are today the major concern of pollution in rivers.

b. Disadvantages

i. In colder climates where the temperature drops to sub normal, the plants may have to be protected with a greenhouse otherwise the biota may freeze up.

ii. Because of higher automation, the technology is not attractive for smaller sizes of plants

iii. The technology requires more qualified operators than in other technologies.

iv. Yet to be validated on reasonable number and sizes of STPs in India

5.18.14.4   Applicability

Organica treatment plants are also on-site sewage treatment systems and the fate of disposal and regrowth of over grown plants is to be addressed with the possibility of biomethanation of the harvested over growths just like the biomethanation of fodder.

5.18.14.5   Guiding Principles

These require to have continuous feed for sustaining the microbes and as such are not recommended for places like hostels, which may lie vacant during holiday seasons and after exam seasons. These may be better for controlled housing colonies without industrial activity but only after an oil and grease trap and with potential for using the treated sewage for avenue trees by dedicated pipeline and root zone drip irrigation but are yet to be validated on reasonable number and sizes of STPs in India.

The toxic or otherwise of the inoculum of 3,000 species and their residues when discharged into aquatic environment like rivers, ponds etc are unknown at this stage and the treated sewage may have to be put through a toxicological clearance from a competent authority before the technology can be taken on board in JnNURM funded STPs.

5.18.15   Submerged Immobilized Biofilm Technology

5.18.15.1   Description

These are stated to be exfoliated bricks of volcanic ash which do not degrade by themselves but offer microbes have a chance to get into the crevices and stay there as immobilized habitats. These microbes bring about the aerobic anaerobic or facultative activity based on prevailing oxygen conditions or septic conditions. These are confined in application to small sized plants and polishing of sewage effluent from STPs. These are stated as patented makes. This can be recommended for outfalls of secondary effluent prior to discharge in the water bodies for polishing of effluents wherever required to meet the discharge standards.

An STP with Eco-Bio-Block (EBB) has been installed and functioning at Indian Institute of Science Education and Research (IISER), Mohali in small scale and the same needs to be evaluated. Further piloting is required. Based on the performance, the same may be recommended for on-site and decentralized wastewater management systems.

5.18.15.2   Application Examples

a. EBB performance test with Ministry of Land, Infrastructure, Transport and Tourism, Nobeoka City, Japan

EBB performance test was carried out by Nobeoka City with Ministry of Land, Infrastructure, Transport and Tourism using the EBB blocks at a river in Nobeoka City.

   i. Study year: 2001

  ii. Test area of river: 2m width, 30m length

 iii. Type of EBB block: EBB 300 (300×300×60mm)

The performance of EBB blocks in treating the drain sewage is mentioned in Table 5.66 overleaf.

Table 5.66 Performance of EBB test in Nobeoka city

	Before EBB	After EBB	
		7days	14days
pH	6.9	6.6	6.5
DO (mg/l)	6.0	7.4	7.5
BOD (mg/l)	2.5	0.7	0.7
COD_{Mn} (mg/l)	8.3	2.7	4.0
SS (mg/l)	110	10	5
Total coliform (10^2) (MPN/100ml)	7.9	4.6	1.3
T-N (mg/l)	1.30	0.39	0.30
T-P (mg/l)	0.34	0.096	0.041
Turbidity	31	5.4	2.8

Source: http://www.ariakeebb.com

5.18.16 BIOFOR Technology (Biological Filtration and Oxygenated Reactor)

5.18.16.1 Key Features of the Technology

• Enhanced primary treatment with addition of coagulants and flocculants

• High rate primary tube settlers and integrated thickening offering space economy

• Two stage high rate filtration through a biologically active media and with enhanced external aeration

• Co-current up flow movement of sewage and enable higher retention and contact

• Treatment scheme excluding secondary sedimentation but recycling of primary sludge

• Deep reactors enabling low land requirements

• A compact and robust system

The process flow diagram of BIOFOR technology is presented in Figure 5.91.
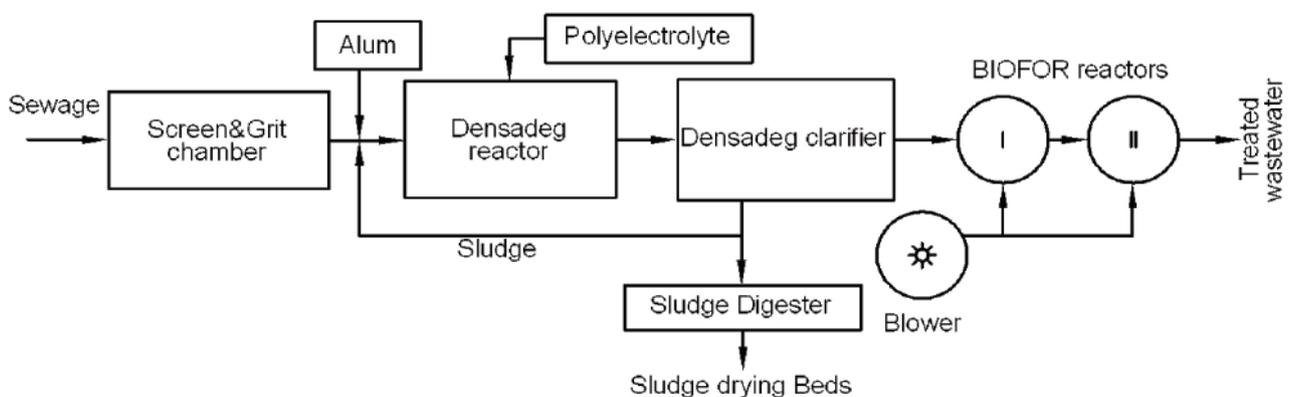
Source: MoUD, 2012

Figure 5.91 Process flow diagram of BIOFOR Technology

5.18.16.2   Advantages and Disadvantages

The advantages and disadvantages of this process are as follows:

a. Advantages

   i.   Compact layout because of high rate processes.

   ii. Higher aeration efficiency through co-current diffused aeration system

   iii. Space saving as secondary sedimentation tank is dispensed

   iv. Ability to withstand fluctuations in flow rate and organic loads

   v.   Compliance with stricter discharge standards

   vi. High quality effluent for reuse without separate nutrient removal and fine filtration

   vii. Effluent suitable for UV disinfection without filtration

   viii. Absence of aerosol and odour nuisance in the working area

   ix. Absence of corrosive gases in the area

   x.   Lower operation supervision enables lesser manpower requirement

b. Disadvantages

   i.   Continuous and high chemical dosing in primary clarification

   ii. Large sludge generation due to the addition of chemicals

   iii. Undigested sludge from primary clarification requiring post treatment

   iv. Yet to be validated on reasonable number and sizes of STPs in India.

5.18.17   High Rate Activated Sludge BIOFOR-F Technology

5.18.17.1   Key Features of this Technology

   • In general, high level of mechanization and sophistication

   • The flow scheme excludes primary sedimentation tank

   • Superior aerated grit chamber and classifier

   • Circular aeration tank with tapered air diffusion system

   • Second stage aeration and rapid sand filtration through a biologically active filter media

   • Dissolved air floatation for sludge thickening

   • Digester heating and temperature controlled anaerobic sludge digestion

   • Mixing of digester contents through biogas

   • Dynamic cogeneration of electrical and thermal energy through gas and dual fuel engines

The process flow diagram of High Rate Activated Sludge BIOFOR-F Technology is presented in Figure 5.92 overleaf.
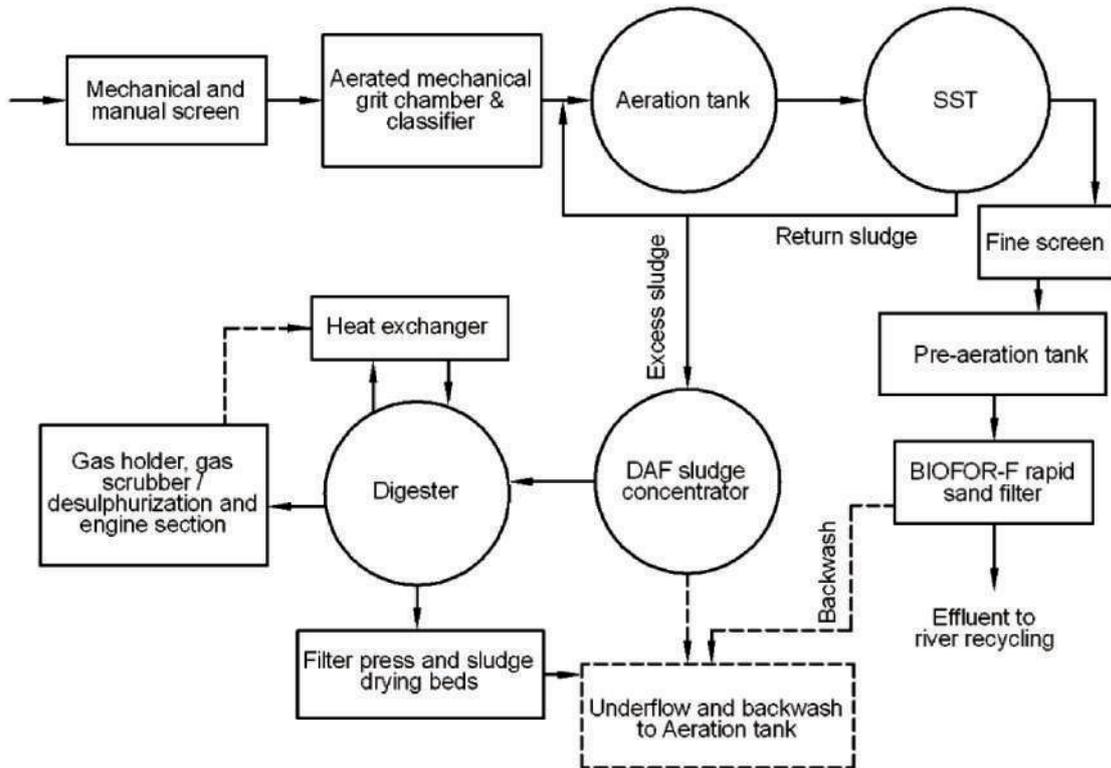
Source: MoUD, 2012

Figure 5.92 Process flow diagram of High Rate Activated Sludge BIOFOR-F technology

5.18.17.2   Advantages and Disadvantages

The advantages and disadvantages of this process are as follows:

a. Advantages

 i. Compact layout as a result of high rate processes

 ii. Higher aeration efficiency through diffused and tapered aeration system

 iii. Space saving as primary sedimentation is dispensed

 iv. Compliance with stricter discharge standards

 v. Effluent suitable for high end industrial applications

 vi. Stable digester performance and consistent gas production

 vii. Almost self-sufficient in energy requirement due to gas engine based cogeneration system

 viii. Absence of aerosol and odour nuisance in the working area

b. Disadvantages

 i. None, except high cost

5.18.18 Submerged Aeration Fixed Film (SAFF) Technology

5.18.18.1 Key Features of the Technology

- Essentially a fixed film media with enhanced oxygen supply through submerged aeration

- Unconventional plastic media offering high void ratio and specific area compared to stone and aggregates

- Large biomass and long solid retention time in the reactor leading to low 'food to micro-organism ratio' and higher organic removal

- Two stage biological oxidation

- Treatment scheme excluding primary sedimentation and sludge digestion

- Reactors up to 6 m deep enabling low land requirements

- Many plants based on such technology are functioning in industrial wastewater applications. Pilot study is required for municipal sewage applications.

Process flow diagram of Submerged Aeration Fixed Film (SAFF) Technology is presented in Figure 5.93.
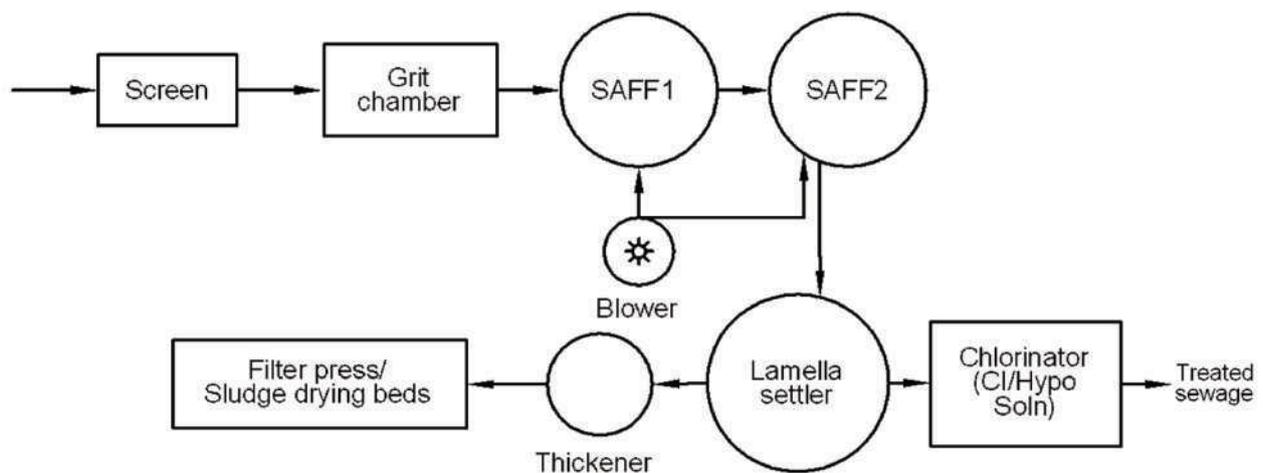
Source: MoUD, 2012

Figure 5.93 Process flow diagram of Submerged Aeration Fixed Film (SAFF) Technology

5.18.18.2 Advantages and Disadvantages

The advantages and disadvantages of this process are as follows:

a. Advantages

  i. Deep reactors enabling small space requirements

  ii. Ability to effectively treat dilute domestic sewage

  iii. Low and stabilised sludge production eliminating the need for sludge digestion

  iv. Absence of odour and improved aesthetics

  v. Absence of emission of corrosive gases.

b. Disadvantages

i. Clogging of reactor due to absence of primary sedimentation

ii. Reliance on proprietary filter media.

iii. Strict quality control on media.

iv. High reliance on external energy input.

v. Requires skilled manpower.

vi. Yet to be validated on reasonable number and sizes of STPs in India

5.18.18.3   Applicability

The SAFF technology based system is particularly applicable for:

- Small to medium flows in congested locations

- Sensitive locations

- Decentralised approach

- Relieving existing overloaded trickling filters.

5.18.19   Rim Flow Sludge Suction Clarifiers

These are clarifiers with inlet along the rim and sludge is sucked out at the floor through suction boxed arms instead of scrappers and is reported to save on foot print and denser sludge and quicker return to aeration tank without lysis of the live sludge.

5.18.19.1   Advantages

- It is claimed that given the same clarifier volume as conventional centre feed clarifiers, these types of clarifiers can handle much higher throughputs and the rising sludge phenomenon is minimized.

- The need for a buried central feed pipe in large central feed clarifiers is avoided.

- The sludge is sucked out as soon as it settles on the floor and transferred to aeration tank and thus avoiding cell lysis.

5.18.19.2   Disadvantages

- Here again, each vendor advocates their own criteria for the equipment and their types which makes it difficult to bring about a common and validated design criteria.

- The sludge suction arrangement if it gets into repair necessitates the emptying of the clarifier for repairs.

5.18.20   Improved Circular Secondary Clarifier (HYDROPLUME®) CSIR-NEERI

The conventional secondary clarifiers do not take hydraulic energy dissipation into account. They are either too large or often fail in giving the efficient solids-liquid separation. In this endeavour, CSIR - NEERI has developed a clarifier design radically different from the conventional circular clarifiers. It is called as HYDROPLUME®, which is an effective hydraulic energy dissipating, solids contact and sludge recirculation type high rate secondary clarifier that provides natural flocculation through plume formation.

It produces excellent effluent quality and helps in attaining the treated effluent quality conforming to discharge standards and the settled sludge is removed through a specially designed suction mechanism. The sludge removal mechanism is designed and fabricated to remove sludge from all around the clarifier and discharging it from a stationary outlet as depicted in Figure 5.94.
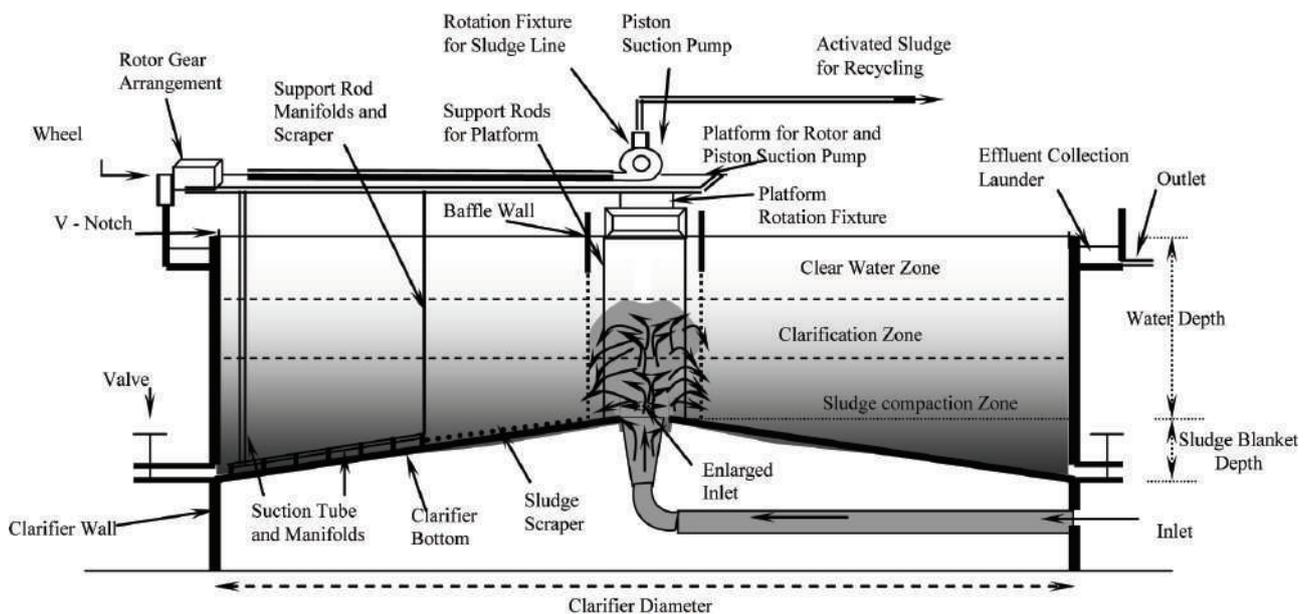
Source: Pophali et al., 2009

Figure 5.94 Sectional Drawing of HYDROPLUME® Clarifier showing the principle of functioning of plume formation, solids-liquid separation and sludge withdrawal (US Patent No. 7637379 B2)

5.18.20.1 Salient Features of HYDROPLUME®

• Hydrodynamics- Optimization of velocity gradient and hydraulic energy dissipation to ensure natural flocculation

• Geometry- Provision of an improved inlet design and bottom to enhance the solids-liquid separation and facilitate sludge removal

• Sludge Removal Mechanism- Development of an improved sludge removal suction mechanism to remove the settled sludge

5.18.20.2   Advantages

• Improved solids-liquid separation ensures minimum SS in the treated effluent

• High underflow solids concentration minimizes pumping rate, and maintains desired active biomass concentration in aeration tank

• Requires less surface area and operates at low hydraulic retention time (1.5 – 2.0 hrs HRT), thereby facilitates savings in capital cost

• It does not require a separate sump cum pump house for sludge recycling / removal and thereby saves capital and recurring costs

• It provides natural flocculation and does not require separate flocculation facility and thereby reduces capital and recurring cost

• The design has been validated using computational fluid dynamics studies for its plume behaviour and hydraulic energy dissipation besides other parameters.

5.18.20.3   Applicability

The next step is to take it up in field scale trials for firming up the design criteria for use in STPs.

5.19   ADDRESSING THE RECENT TECHNOLOGIES IN CHOICE OF STP

A reference is made to the advisory on recent trends in technologies in sewerage system issued by the MoUD in March 2012, to encourage the implementers in the field to innovate and explore new technologies as well as Public Private Participation (PPP) models without compromising on the basic safeguards both technical and financial. It states that, "There are various technology options available for treating sewage. The technology option as well as the project cost would be outlined in the detailed project report prepared for implementing the project. Irrespective of the technology chosen, STP projects could be developed on a long term commitment from the private sector partner either on PPP / build own operate transfer (BOOT) basis or on engineering procurement construction (EPC) plus O&M for 15 years where a part of the EPC cost is payable over a long-term O&M period. However, it is suggested that no new technologies will be considered under EPC contract".

Box No. 5.1   Honouring Edward Ardern, MSc and William T. Lockett, MSc

It was on April-3-1914 that Edward Ardern, MSc and William T. Lockett, MSc, presented their paper titled "Experiments on the Oxidation of Sewage without the Aid of Filters" (at the Society of Chemical Industry in Manchester, England), in which they made the first reported use of the term "activated sludge" to refer to biological solids that they settled out of aerated sewage and recycled these back into the treatment process. Almost all the aerobic biological treatment processes to date, trace their lineage to this epoch making invention of the Activated Sludge Process. They continued to lead distinguished research and are reported to have published over 25 papers.

This manual honours them in the centenary year of 2014.