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भारत सरकार
आवासन और शहरी कार्य मंत्रालय
निर्माण भवन
GOVERNMENT OF INDIA
MINISTRY OF HOUSING AND URBAN AFFAIRS
NIRMAN BHAWAN

नई दिल्ली–110011, तारीख 20
New Delhi-110011, dated the 20

Dated : 08/07/2024

DO No.Z.16025/3/2024-CPHEEO

Dear Sir/Ma'am,

Used Water Management is an important component under SBM-U 2.0. With States/ULBs pro-active efforts about 85% of City Sanitation Action Plans have been approved by the Ministry till date. As we are going to complete 3rd year of Mission period as on 1st October, 2024, the expectations from States and ULBs is to put all efforts for expeditious implementation of Action Plans within Mission Period.

In the course of inter-action with ULBs/States, at various forums, CPHEEO has identified requirements for guiding ULBs on various technologies, which are key for economical and sustainable treatment of used water. Accordingly, CPHEEO has prepared **"Draft Type designs of 1,2 and 5 MLD capacity STPs"** which are tested for performance and are widely used. In addition, the technology cost specific analysis, lifecycle cost assessment (LICCA) is also included in the draft type designs.

States are requested to circulate above documents to the concerned ULBs and their parastatal agencies/technical wings to examine and share suggestions/comments, if any, for further improvement latest by 20th July, 2024 at email Deepak@washinstitute.org.

With regards,

Yours sincerely,

(Dr. V.K. Chaurasia)

To,

SMDs of all States/UTs
MDs of all parastatals.

Copy to: PPS to JS(SBM) for information.

Swachh Bharat Mission - Urban

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Advisory on

Type Design of STPs for Small and Medium Towns

Central Public Health and Environmental Engineering Organization (CPHEEO)

MINISTRY OF HOUSING & URBAN AFFAIRS

Government of India

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Preface

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Acknowledgement

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Executive Summary

The Ministry of Housing & Urban Affairs is implementing SBM 2.0, since 1st October 2021 with a key objective of preventing the disposal of any untreated Used Water into the environment. One of the crucial components eligible for funding under the Mission is Used Water Management (UWM). The focus under UWM component is, Interception & Diversion of drains carrying untreated Used Water and to develop sewage treatment systems in 4000+ smaller ULBs across the country, having populations less than 1 lakh.

However, many of these smaller ULBs lack sufficient capacity to plan, design, implement & maintain the complex Used Water Management facilities. To address this challenge and execute safely managed sanitation practices, Ministry is undertaking various initiatives to build the capacity of officials of such ULBs. This is essential for ensuring all the UWM proposals under SBM 2.0 are completed on time & to be sustainable at the same time, thereby eliminating the practice of unsafe disposal of untreated Used Water into the environment, as such practices will lead to significant health, environmental, and socio-economic issues

These advisory aims to help State/ULBs officials choose suitable technologies that match their needs, available resources, and the specific site conditions. This advisory has been prepared by taking into account the current capacities of State/ULBs officials and the implementation timeframe of the Mission. This document enables ULB officials to directly adopt designs and cost estimations (Land cost, CAPEX, & OPEX) of 10 commonly used/combination technologies, with minor adjustments as per their requirements, thereby expediting the implementation of the Mission and ensuring its long-term sustainability

This advisory is structured into four chapters:

1) Background,
2) Design calculation
3) Cost estimates & Life cycle cost assessment
4) Used / Grey wate management in Class V & VI Towns

each addressing critical aspects of Used Water management.

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The **First chapter** offers a comprehensive overview of the unique aspects of SBM-U 2.0, emphasizing the innovative shift from using the term "Wastewater" to "Used Water." This terminological change underscores the initiative's forward-thinking approach. The chapter clearly outlines the UWM approach, objectives of the advisory, establishing a foundation for a holistic understanding of the initiative. Additionally, this chapter details the permissible project components under SBM-U 2.0 and the funding mechanisms available, including grants to State and Urban local bodies (ULBs) through the 15th Finance Commission. It also elaborates on the effluent parameters essential for designing sewage treatment plants (STPs) and provides **clarification on the National Green Tribunal (NGT) order regarding wastewater discharge standards**. The advisory considers 10 specific STP technologies combinations for design and cost estimation, as below:

- Waste stabilization pond
- Anaerobic Baffled Reactor (ABR) + Constructed Wetlands (CWL)
- Upflow Anaerobic Sludge Blanket (UASB) Reactor + WSP
- UASB Reactor + Activated Sludge Process (ASP)
- Sequential Batch Reactor (SBR)
- Extended Aeration (EA)
- Moving Bed Biofilm Reactor (MBBR)
- Trickling Filter (Bio-Tower)
- UASB+Aerated Facultative Ponds
- Anaerobic Pond + Aerated Facultative Ponds

Further, along with Design & cost estimation of above technologies Preliminary treatment & Primary treatment units- like Screens, Grit chamber & Main pumping station, Primary & secondary clarifier & chlorination tank and also design.

The **Second chapter** on Design Calculations clearly explains the workings of various treatment methods and their mechanisms. It provides detailed calculations for designing a plant capable of treating 2 MLD of wastewater, considering specific site requirements and assumptions drawn from standard documents like the CPHEEO manual and IS codes. These guidelines

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ensure that treatment units are appropriately sized and configured to meet local environmental and project requirements

The **Third chapter**, focusing on Life Cycle Cost Assessment, presents thorough cost estimates for sewage treatment plants (STPs) of 1, 2, and 5 MLD capacities. It includes three main costs- Capital costs, Operation and Maintenance costs, and land costs, considering various scenarios such as land costing 0 (assuming land available in ULBs), 10, 25, 50, 75, 100 lakh/acre. The chapter also features a **Comparative Cost Assessment Matrix** for 10 technologies, illustrating total project costs under different land cost scenarios. This assists state ULBs in selecting cost-effective technologies over the system's lifespan, ensuring sustainable management of Used Water treatment system

The **Comparative Cost Assessment Matrix or Life Cycle Cost Assessment** shows that non-mechanized technologies have significantly lower overall costs, including expenses for land, capital, and operation & maintenance to each technology or combination. In contrast, mechanized treatment systems incur higher total costs for smaller STPs, as detailed in the advisory on page 271 titled "STP Project Life Cycle Cost Assessment. Additionally, it came to in notice that the land requirement for non-mechanized technologies are higher compared to the Mechanized technology.

In this situation, planners and designers are challenged with carefully assessing current requirements. They need to decide whether to prioritize adopting mechanized or non-mechanized technologies, or to focus on acquiring land.

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Comparative Cost Assessment Matrix (STP Project Life Cycle Cost)

Sl. No.	Technologies	Zero Land Cost (Rs. In Lakhs)			10 Lakhs Land Cost (Rs. In Lakhs)			25 Lakhs Land Cost (Rs. In Lakhs)			50 Lakhs Land Cost (Rs. In Lakhs)			75 Lakhs Land Cost (Rs. In Lakhs)			100 Lakhs Land Cost (Rs. In Lakhs)		
		1 MLD	2 MLD	5 MLD	1 MLD	2 MLD	5 MLD	1 MLD	2 MLD	5 MLD	1 MLD	2 MLD	5 MLD	1 MLD	2 MLD	5 MLD	1 MLD	2 MLD	5 MLD
1	Waste Stabilization Ponds (WSP)	850	1141	1931	1060	1508	2777	1376	2058	4045	1903	2975	6160	2429	3892	8275	2955	4809	10390
2	Anaerobic Bafled Reactor (ABR) + Constructed Wet Lands (CWL)	1318	2194	4963	1472	2486	5672	1704	2925	6735	2091	3656	8506	2477	4387	10277	2863	5118	12048
3	Up Flow - Anaerobic Sludge Blanket Reactor+ WSP (UASB + WSP)	1071	1640	2888	1330	1970	3648	1499	2465	4789	1927	3290	6689	2355	4115	8590	2783	4940	10490
4	Up Flow - Anaerobic Sludge Blanket Reactor + ASP (UASB + ASP)	2110	2717	4261	2131	2743	4296	2163	2783	4348	2216	2850	4434	2269	2917	4521	2323	2984	4608
5	Sequence Batch Reactor (SBR)	3045	4223	7364	3066	4246	7400	3097	4280	7454	3149	4338	7544	3201	4395	7634	3254	4453	7724
6	Extended Aeration Process (EAP)	2780	3564	6222	2802	3593	6269	2835	3636	6340	2889	3708	6458	2943	3781	6576	2997	3853	6694
7	Moving Bed Bio Reactor (MBBR)	2123	2811	5073	2141	2834	5106	2168	2869	5156	2213	2928	5240	2258	2986	5323	2303	3045	5407
8	Anaerobic Pond + Aerated Facultative Pond (AP+AFP)	913	1300	2427	1081	1590	3099	1330	2018	4086	1752	2744	5767	2174	3471	7447	2596	4198	9127
9	Upflow Anaerobic Sludge Blanket Reactor + Aerated Faculative Pond (UASBR + AFP)	1108	1661	3109	1337	1935	3748	1461	2348	4707	1814	3035	6304	2167	3722	7902	2520	4409	9500
10	Bio Tower (Tricking Filter)	1672	2610	4805	1690	2631	4837	1717	2662	4885	1762	2715	4964	1807	2767	5043	1852	2819	5123

Technology	Capacity	Land cost (Rs. / acre)
Anaerobic Pond + Aerated Facultative Pond	1 MLD, 2 MLD, 5 MLD	Rs. 0, 10, 25, 50
Bio-Tower (Trickling Filter)	1 MLD, 2 MLD	Rs. 75 lakhs and Rs. 100 lakhs
UASB Reactor + Activated Sludge Process	5 MLD	Rs. 75 lakhs and Rs. 100 lakhs
NOTE	The technology with lowest LCC for each combination of capacity and land cost is highlighted in green. Alternate technology very close to the lowest LCC technologoy is highlighted in blue. Technology with the highest LCC is heighted in Red.	
Conclusion	For all STP capacities up to Rs. Rs. 50 lakhs / acre land cost, (UASBR + AFP) has emerged as the least LCC technology and SBR as the highest LCC technology. For land cost of Rs. 75 and 100 lakhs/acre Bio-Tower is lowest LCC option up to 2 MLD and UASBR + ASP for 5 MLD.	

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The **Fourth chapter** focuses on Grey Water Management Strategies design for **Class V and VI towns,** addressing both household and community levels of Used Water management. It suggests treatment systems for Household Used Water management, which can originate from septic tanks , with capacities of 0.5, 1, and 2 KLD, providing estimated sizes and costs.

Similarly for Community level treatment, it provides size options and cost projections for community-scale treatment facilities designed to manage wastewater volumes ranging from 3 to 10 kilolitres per day (KLD). For higher capacities, like 100-500 KLD in community-level wastewater management, the suggested treatment approach includes intercepting and redirecting drainage flows to preliminary treatment stages, followed by treatment in waste stabilization ponds or constructed wetlands, and ultimately, discharge management.

These options help ULBs choose appropriate treatment systems, promoting sustainable and decentralized approaches that benefit smaller towns and areas.

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CHAPTER – 4 USED / GREY WATER MANAGEMENT IN CLASS V AND CLASS VI TOWNS

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CHAPTER – 1
BACKGROUND

1.1 Uniqueness of SBM-U 2.0

The Swachh Bharat Mission-U 2.0 was launched by the Hon'ble. Prime Minister of India on October 1, 2021. Having achieved the goal of making the country ODF under SBM launched in 2014, the goal of SBM-U 2.0 is to make cities garbage-free. On the occasion of the launch of the Mission, the Hon'ble Prime Minister rightly echoed the objectives of the Mission in the following words:

"We have to remember that cleanliness is not a task just for a day, a fortnight, a year or for just a few people. Cleanliness is a great campaign for everyone, every day, every fortnight, every year, generation after generation. Cleanliness is a lifestyle, cleanliness is a life mantra."

Of special importance is the introduction of 'Used Water Management' component under SBM-U 2.0 which is a unprecedented move which is likely to benefit every non-AMRUT ULB (towns with less than 1 lakh population) in the country. This component has been introduced to address the deficiency of sewerage system under which about 50% of urban population depends on unregulated on-site sanitation systems.

1.2 Used Water Management

At the outset, it is necessary to highlight the terminology employed by SBM-U 2.0; 'Used Water' in place of 'Wastewater'. This is likely to register a major paradigm shift in the sub-conscious mind of the urban planners as well as the dwellers, and propel them towards recognizing the fact that Used Water is not waste water since it can be treated and reused. It is estimated that about 13000 MLD of Used Water can be treated and made available for non-potable reuse purposes in Class II to Class VI towns of the country. This is for the first time that the government has given importance to treatment of Used Water in small towns at par with Class I and metropolitan cities. As per the statement in the SBM-U 2.0 Operational Guidelines – 2021, about 10.42 Crore people are likely to benefit in 3, 901 cities of the country.

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1.3 Admissible Project Components and Funding under SBM – U 2.0

The project components related to sewerage system are to be funded from the Central and State Government / ULB funds. The eligibility of project components for funding under the two streams of funds is shown below in Figure 1.1.

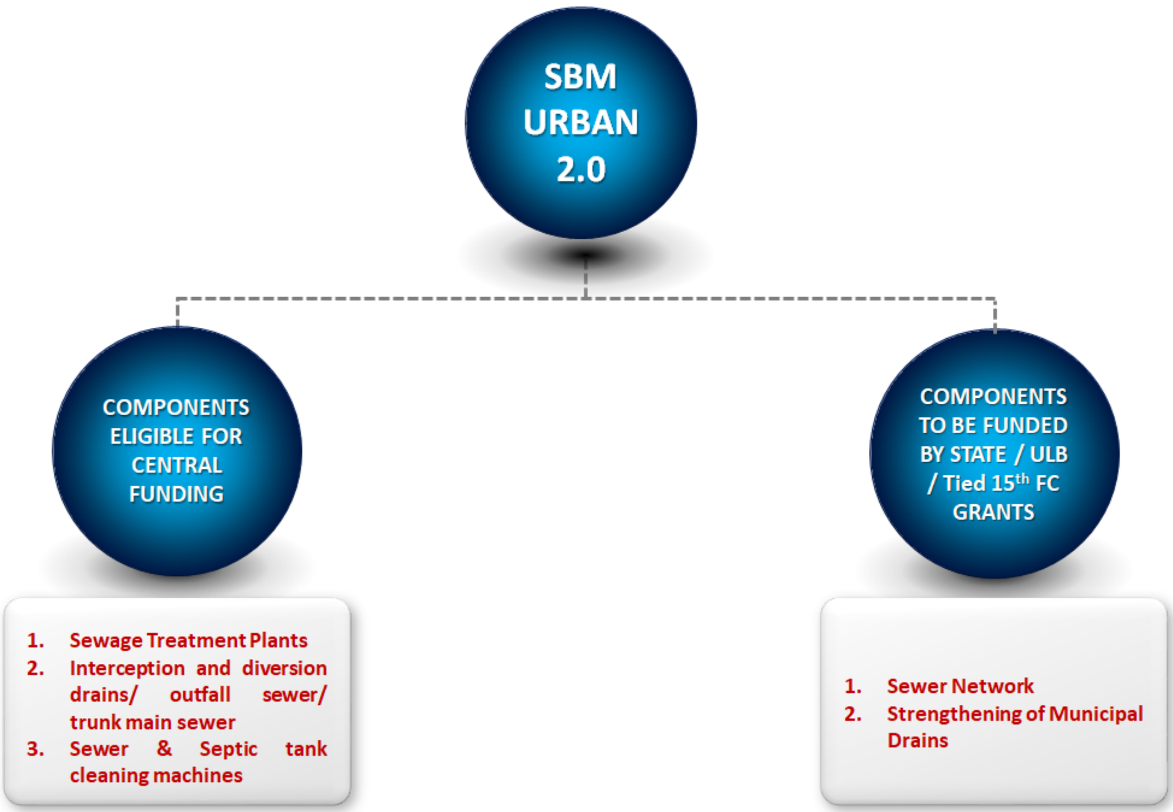

Figure 0.1 Components of Sewerage System eligible for funding under SBM-U 2.0

1.4 Approach for Used Water management & FSM

To amicably address used-water management challenges under SBM 2.0, an integrated plan for a city is to be prepared involving sewered and non-sewered solutions including faecal sludge management and the implementation maybe taken up as per the availability of resources and priority in an incremental approach. However, in some states and cities where already a significant improvement is done in used-water management, the same maybe taken up to saturation level. The detailed approach are as follows:

Sewer Network in Core Sanitation Zone: ULBs have to identify its “Core Sanitation Zone (CSZ)”, defined as a zone which has at least 50% of the town's current population settled over an area comprising about 20-30% of the town's spread (please refer to the diagram given below). The CSZ will be provided with a sewer network to connect it directly to the STP. The cost of the CSZ sewer network will be borne entirely by the State/ ULB from 15thFC Grants/ SFC Grants/ their own funds etc. States/ UTs are expected to encourage the ULBs to identify any suitable area in the city to provide a sewer network. City can expand network coverage based on necessity and availability of resources over time. For upcoming new green field developments in and around towns, the provision of sewerage network along with decentralized sewage treatment facilities should be factored into planning.

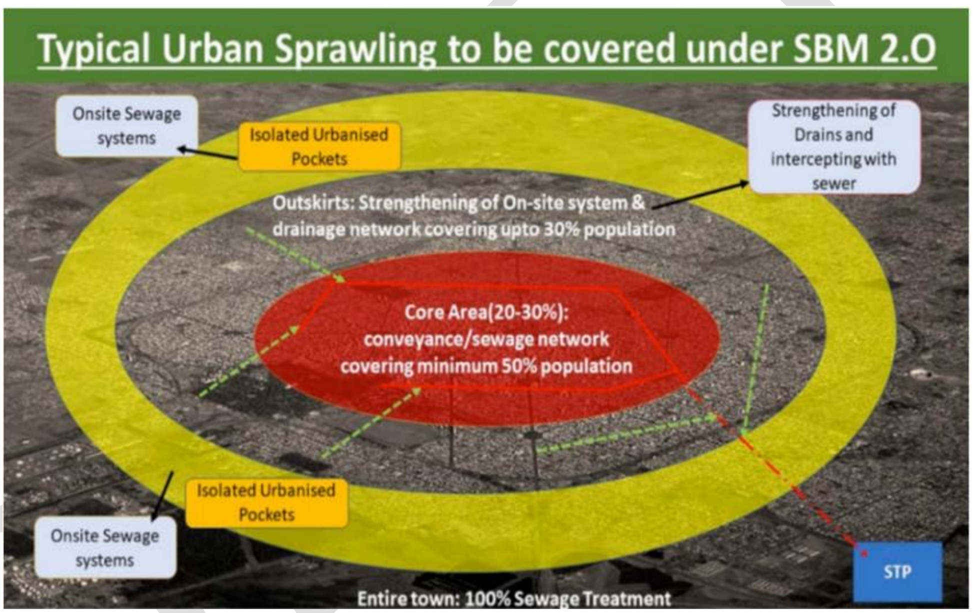

Figure 2: Urban sprawling to be covered under SBM 2.0

Intercepting Used Water from open drains to Sewer network: State is also required to strengthen existing open drains carrying sullage/sewage and connect the same to the sewer network, wherever feasible, after providing suitable I&D structures like coarse screen, grit chamber, fine screen and settling basin etc. before intercepting into sewer network.

Approach for Fringe Areas: For remaining inhabitants residing in fringe areas outside the CSZ, the town authorities may work out economically judicious solutions, opting between continuing with onsite disposal systems (septic tanks with soak pits) and providing localized community level sewage treatment

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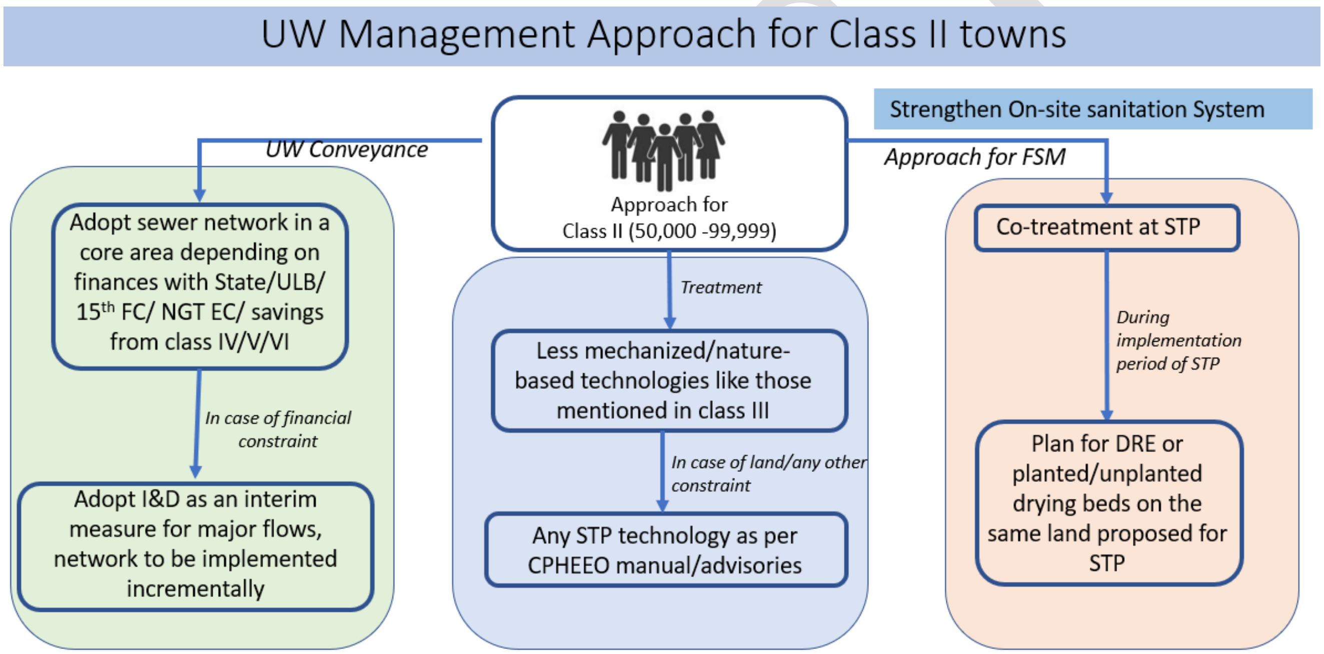
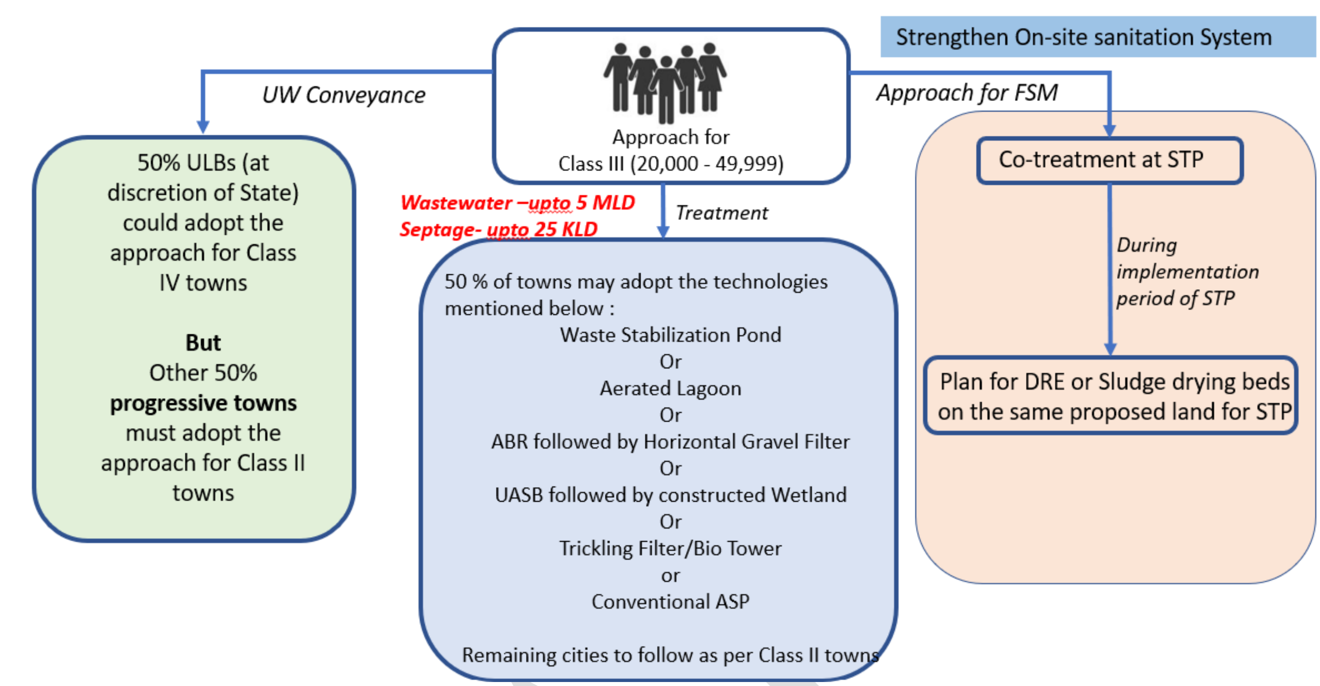
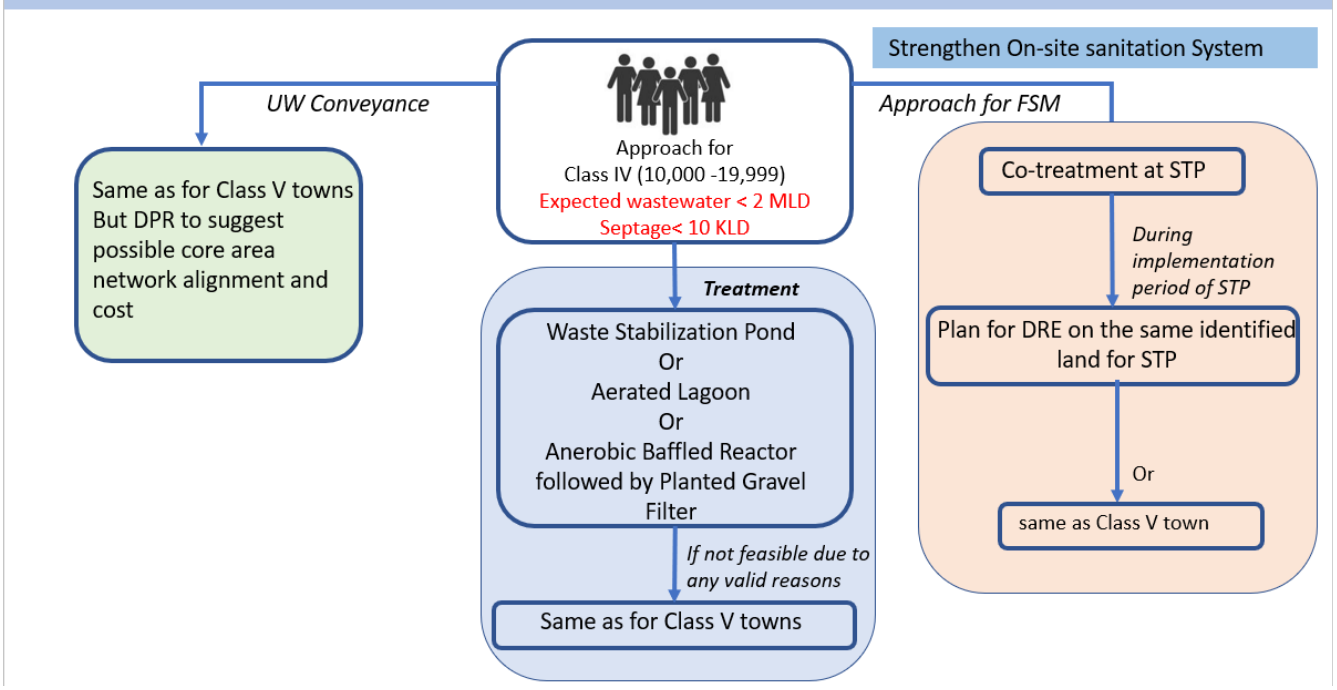
plants for grey/ black water where feasible or conveying it to STP depending on economics. The septage from these households will continue to be safely hauled to a designated STP under professional arrangements. It is advised that the fringe areas may try to strengthen their onsite disposal arrangements by providing for soak pits where they are missing and forcing the septic tank effluent into the ground, adhering to IS 2470. However, due to practical difficulties in providing sewerage systems to the entire population, onsite sanitation systems are encouraged in the fringe areas of the towns where it is uneconomical to provide sewers and in areas where it is difficult to provide sewer network. In such cases, STPs along with co-treatment of faecal sludge have been proven to be advantageous and sustainable.

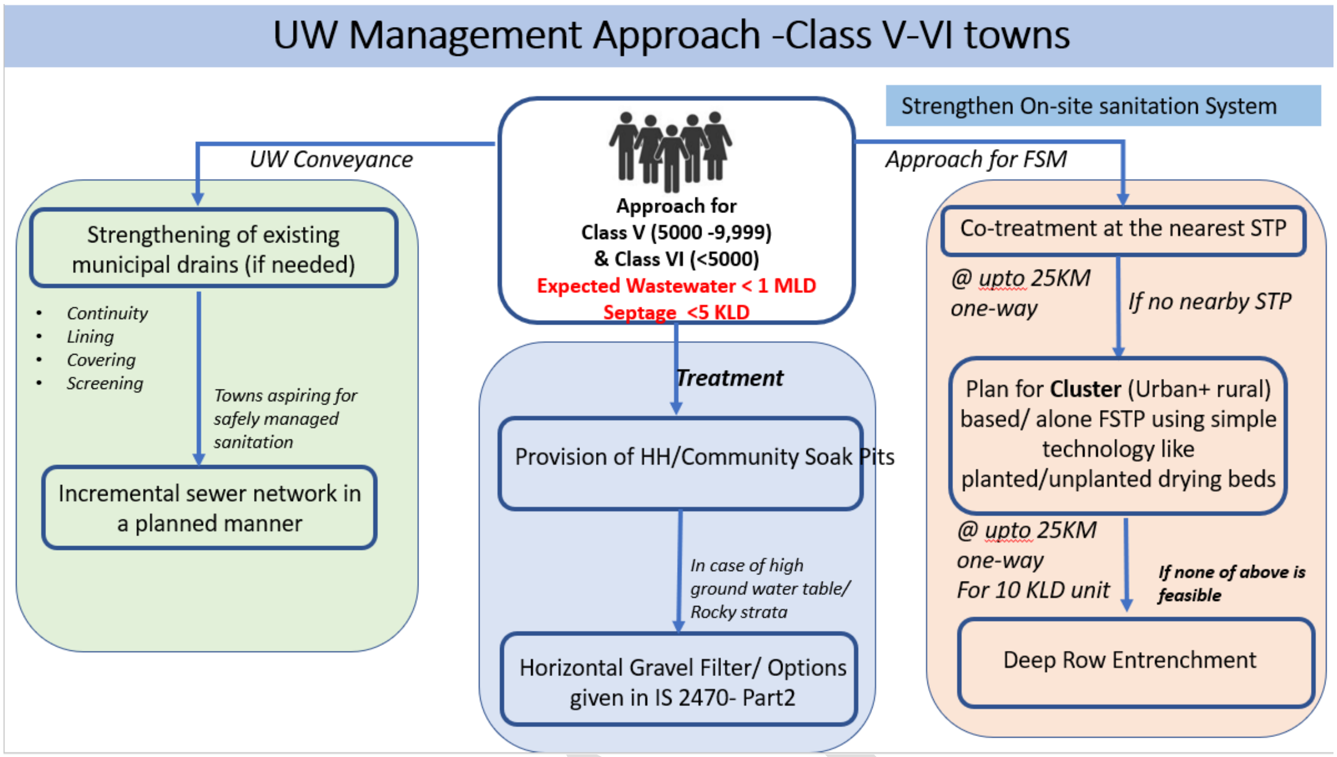


UW Management Approach for Class III towns



UW Management Approach for Class IV towns




UW Management Approach -Class V-VI towns
Strengthen On-site sanitation System
UW Conveyance
Approach for FSM
Approach for
Class V (5000 -9,999)
& Class VI (<5000)
Expected Wastewater < 1 MLD
Septage <5 KLD
Strengthening of existing municipal drains (if needed)
Continuity
Lining
Covering
Screening
Towns aspiring for safely managed sanitation
Incremental sewer network in a planned manner
Treatment
Provision of HH/Community Soak Pits
In case of high ground water table/ Rocky strata
Horizontal Gravel Filter/ Options given in IS 2470- Part2
Co-treatment at the nearest STP
@ upto 25KM one-way
If no nearby STP
Plan for Cluster (Urban+ rural) based/ alone FSTP using simple technology like planted/unplanted drying beds
@ upto 25KM one-way For 10 KLD unit
If none of above is feasible
Deep Row Entrenchment

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1.5 Sewage Treatment Process

A Sewage Treatment Plant (STP) is a facility designed to remove contaminants from wastewater and household sewage, both effluents and domestic. Its purpose is to produce an environmentally safe fluid waste stream (or treated effluent) and a solid waste (or treated sludge) suitable for disposal or reuse. Here's a breakdown of how an STP works:

1. **Preliminary Treatment**: This involves the removal of large solids and debris through screening and grit removal. It's the first step to protect the subsequent treatment processes from damage and clogging.
2. **Primary Treatment**: In this stage, wastewater is allowed to settle in large tanks, causing heavier solids to sink to the bottom as sludge and lighter materials such as grease and oils to float to the top. These materials are then removed, reducing the load on the secondary treatment process.
3. **Secondary Treatment**: This step involves biological processes where microorganisms break down organic matter in the wastewater. This is typically achieved through aeration (adding air to promote microbial growth) in activated sludge systems or through biofilms in trickling filters.
4. **Tertiary Treatment**: This is an additional purification process that further reduces contaminants in the effluent. It can include processes like filtration, lagooning, nutrient removal, and disinfection (using chlorine, UV light, or ozone) to ensure the treated water is safe for discharge into the environment or for reuse.
5. **Sludge Treatment**: The solids collected from the primary and secondary treatments are further processed to reduce volume and pathogens, often through anaerobic digestion, composting, or incineration. The resulting stabilized sludge can be used as fertilizer or disposed of safely.

1.6 Fecal Sludge Treatment plant

A Fecal Sludge Treatment Plant (FSTP) is a facility designed to treat fecal sludge, which is the semi-solid waste collected from on-site sanitation systems such as pit latrines, septic tanks, and other forms of non-sewered sanitation. The purpose of an FSTP is to safely process and treat

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fecal sludge to produce treated effluent and solid by-products that can be safely disposed of or reused. Here’s a detailed explanation of how an FSTP works:

1. **Collection and Transportation:** Fecal sludge is collected from on-site sanitation facilities using vacuum trucks or manual methods. It is then transported to the FSTP for treatment.
2. **Screening:** Upon arrival at the FSTP, the fecal sludge is unloaded at a reception area where large debris and non-biodegradable materials are removed through screening. This step prevents damage and clogging in subsequent treatment processes.
3. **Dewatering of Sludge:** The sludge is then directed to Planted or Unplanted Sludge drying beds where solid particles are get separated through the liquid fraction. The Dewatering process separates the solid and liquid components, concentrating the solids.
4. **Primary Treatment:** In primary treatment, the thickened sludge undergoes further processing to reduce its volume and stabilize it.

   Composting: Sludge is mixed with organic materials and composted to produce a stable, nutrient-rich product that can be used as a soil conditioner.

5. **Secondary Treatment:** The liquid effluent pass through the filter beds can be go for further treatment using DeWats system such as Anaerobic baffled reactor, Constructed wetland.
6. **Tertiary Treatment:** For higher quality effluent, tertiary treatment processes such as disinfection (using chlorine) and nutrient removal may be employed to ensure the treated water is safe for discharge or reuse**.**
7. **Final Disposal or Reuse**

   Solid By-Products: The dewatered sludge can be used as compost or soil conditioner in agriculture, or safely disposed of in landfills.

   Treated Effluent: The treated water can be discharged into water bodies, used for irrigation, or further treated for other uses.

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1.7 Comparision of STP & FSTP

STPs are designed to treat Used Water from **centralized sewer systems**, which includes domestic sewage & other types of Used Water forms, managing continuous flow with a multi-stage process involving biological and chemical treatments.

In contrast, FSTPs handle batch inputs of semi-solid waste from **non-sewered sanitation systems** like pit latrines and septic tanks, focusing on sludge dewatering/stabilization through processes like Filtration, sedimaentation, & composting.

In addition to these two treatment systems, another option is co-treatment, where fecal sludge management is integrated into existing STPs instead of constructing separate FSTPs. This approach maximizes the use of established infrastructure and operational frameworks to effectively manage both centralized sewer system sewage and semi-solid waste from decentralized sanitation systems.

Co-treatment strategies improve resource efficiency and streamline wastewater and fecal sludge management practices in urban and peri-urban areas. This integrated approach promotes sustainable and cost-effective sanitation solutions, mitigating environmental impact and advancing public health initiatives.

1.8 STP Technologies

State / ULB is free to adopt any proven technology, as brought out in the CPHEEO Manual / MoHUA Advisories from time to time.

i. Waste Stabilization Ponds (WSP)
ii. Anaerobic Baffled Reactor (ABR) + Constructed Wetlands (CWL)
iii. Upflow Anaerobic Sludge Blanket (UASB) Reactor + Waste Stabilization Ponds (WSP)
iv. Upflow Anaerobic Sludge Blanket (UASB) Reactor + Activated Sludge Process (ASP)
v. Sequential Batch Reactor (SBR)
vi. Extended Aeration (EA)
vii. Moving Bed Biofilm Reactor (MBBR)
viii. Trickling Filter (Bio-Tower)
ix. UASB + Aerated facultative ponds
x. Anaerobic pond + Aerated facultative ponds

However, for smaller ULBs, nature-based technologies in suitable combinations may be adopted.

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As per the Operational Guidelines of SBM-U 2.0, the ULB / State Government is empowered to select any proven technology as brought out in the CPHEEO Manual/ Advisories from time to time. In case States come across any other technology not listed in CPHEEO Manual / Advisories, the same should be referred to CPHEEO for evaluation and inclusion in the Advisories. State Governments are encouraged to select nature-based sewage treatment technologies (alone or in combination of two to attain desired treated effluent quality), where feasible, to economise CapEx & OpEx.

Relevant Manuals / Advisories on STP Technologies

1. Manual on Sewerage and Sewage Treatment Systems, 2013, CPHEEO, MoH&UA, GoI.
2. Ready Reckoner on Municipal Used Water Treatment Technologies for Medium and Small Towns, CPHEEO, MoH&UA, GoI.
3. Swachh Bharat Mission – Urban 2.0, Operational Guidelines, October 2021, MoHUA, GoI

1.9 STP Effluent Quality

The treated effluent quality influences the outcome of techno-economic analysis which is an integrated process and cost mode. Lack of clarity on the effluent standards to be adopted leads to either over-treatment leading to unnecessary financial burden or under-treatment leading to degradation in the quality of receiving water bodies. The effluent quality norms presented in Table 1.1 are adopted from Table 5.3 of CPHEEO's 'Manual on Sewerage and Sewage Treatment Systems - 2013'.

Table 1. 1 Standards for STP effluents from CPHEEO's manual

S. No.	Quality Indicator	Inland Surface Water	Units
1	Biochemical Oxygen Demand	30	mg/l
2	Chemical Oxygen Demand	250	mg/l
3	Total Kjendahl Nitrogen (TKN) as (N)	100	mg/l
4	Ammonical Nitrogen NH_3-N	50	mg/l
5	TSS	100	mg/l
6	Faecal Coliform, MPN / 100 ml	1000	mg/l

The above effluent standards are adopted in the present study in light of the following comment in the 'Ready Reckoner on Municipal Used Water Treatment Technologies for Medium and Small Towns, CPHEEO':

The 'general discharge standards 1986', were revised vide MoEF&CC notification dated 13th October 2017 in respect of few important parameters. Subsequently, Hon'ble NGT stayed the notification and directed MoEF&CC vide OA no.1069/2018 dated 30th April 2019, to issue an appropriate Notification in the matter. But since the matter is subjudice and revised standards yet not notified by MoEF&CC, therefore, the 'general discharge standards 1986' still prevails except in those cases where CPCB/ SPCB enforced a more stringent set of standards.

1.10 Objectives of the Advisory

Used Water Management is an important component under SBM – U 2.0. The objective of inclusion of Used Water management under SBM-U 2.0 is to ensure that all Used Water is safely collected, treated and reused to feasible extent and no untreated Used Water is discharged into water bodies or the open environment. In order to achieve this objective, the Mission has made provision under central share of funding for setting up of sewage treatment plants and laying of I & D structures / pumping stations / pumping mains or gravity mains up to STPs.

It is expected that a major part of the allocation for Used Water management will be used for construction of STPs. Since SBM-U 2.0is for class II to class VI towns, it is pertinent to note that these small towns with population of less than 1.0 lakh have been empowered for the first time to treat and reuse / manage their Used Water.

The STP technologies which have been popularized and adopted over the past two decades, and with which the ULB staff and consultants are conversant with, have proved to be cost-effective for Mega-Metropolitan cities, Metropolitan cities and class A cities. The main thrust of these technologies has been to minimize the foot print of the STP as land availability and its cost are major constraints in large cities. Technologies with small foot print are usually found to be more energy intensive in their operation, leading to higher O & M cost. In large cities

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where land costs are prohibitively high, technologies with high O & M cost tend to exhibit a lower life cycle cost in spite of being emergency intensive.

The present study is therefore undertaken with the following objectives:

a. To study in detail the techno-economic feasibility of low-cost natural technologies with large footprint vs. energy intensive low footprint technologies based on life cycle cost analysis. The selected technologies (combinations) are as follows:

    i. Waste Stabilization Ponds (WSP)
    ii. Anaerobic Baffled Reactor (ABR) + Constructed Wet Lands (CWL)
    iii. Upflow Anaerobic Sludge Blanket (UASB) Reactor + Waste Stabilization Ponds (WSP)
    iv. Upflow Anaerobic Sludge Blanket (UASB) Reactor + Activated Sludge Process (ASP)
    v. Sequential Batch Reactor (SBR)
    vi. Extended Aeration (EA)
    vii. Moving Bed Biofilm Reactor (MBBR)
    viii. Trickling filter (Bio-Tower)
    ix. Anaerobic pond + Aerated facultative Pond
    x. Upflow Anaerobic Sludge Blanket (UASB) Reactor + Aerated facultative Pond

b. To design and undertake cost estimation of CAPEX, OPEX and land cost components leading to life cycle cost analysis of each technology and its combinations.

c. Develop a general chart which can be used to select appropriate technology based on STP capacity and prevailing land cost in the ULB.

d. To provide guidance to ULBs on managing used water in Class V and VI towns, including details on unit sizes and cost estimation.

Clarification on Effluent standards to be adopted

In accordance with the effluent quality standards outlined in Section 1.9, the rationale for adopting these specific standards is clarified below:

Another important factor that needs to be considered in technology selection is the STP effluent quality. ULBs and consultants who are involved in preparing DPRs for STPs are found to be inclined towards adopting the standards recommended by NGT in its order dated 30-04-

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2019 irrespective of keeping in view of population of Towns.. Under this direction, the STPs are being designed for the effluent parameters listed in Table 1.2.

Table 1.2 STP effluent standards from NGT order dated 30-04-2019

Parameter	BOD	TSS	COD	Total N	Total P	FC Desirable
Units	**mg/l**	**mg/l**	**mg/l**	**mg/l**	**mg/l**	**MPN/100 ml**
Mega and Metropolitan Cities	10	20	50	10	1.0	100

In the recent hearing on 23-03-2023 before the National Green Tribunal Principal Bench, New Delhi, pertaining to compliance of Municipal Solid Waste Management Rules 2016 and other environmental issues in respect of State of Uttar Pradesh, the Tribunal passed the following judgement under 67 (ii):

67. Based on interaction with States/UTs extensively on the issue of solid and sewage waste management, we are of the view that Central Ministries and Departments need to facilitate States/UTs to effectively execute centrally sponsored projects. This will include utilization of waste for defined purposes involving components of central funding. Some such aspects include (i) utilisation of installed STPs are fully utilized remaining unutilised due to lack of connectivity which can be overseen by MoUD. Utilization for treated sewage should be taken as an integral part of the sewage treatment planning with STPs. ***(ii) looking into applicability of standards for sewage treatment in Urban and Rural areas, considering the usage of treated sewage and mode of disposal under the Water (Prevention and Control of Pollution) Act. 1974. This can be done by MoUD, MoEF&CC and CPCB under the coordination of MoUD;*** *(iii) maximizing use of treated sewage and the compost made out of municipal solid waste as full or partial substitute of fertilizer and ultimately reviewing subsidy issue which may be done under joint coordination of MoUD and Ministry of Agriculture and Ministry of Chemical and Fertilizer (iv) process of setting up of waste to energy projects as per applicability in cities and towns with specified technologies and ensuring compliance with environmental norms by Ministry of Power and Ministry of Non-Renewal Energy (MNRE). We have already cleared that such projects may be kept out of the scope of environmental clearances but taking due care based on siting and preventing human health damages (v)*

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specific directions on management of rejects out of biomining processes of legacy waste to avoid haphazard disposal/dumping by CPCB and MoEF&CC.

In light of the above judgement of NGT and the comment made in the 'Ready Reckoner on Municipal Used Water Treatment Technologies for Medium and Small Towns, CPHEEO', it is deemed that municipal sewage may be treated as per those indicated in section 1.9, especially for Small & Medium Towns.

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CHAPTER – 2

DESIGN CALCULATIONS

2.1 Technologies considered for Small and Medium Towns

Considering the urgency of preventing pollution of inland water bodies and preserving their ecological balance, treatment of Used Water prior to its disposal or reuse, needs to be accorded a very high priority. There are several technologies of Used Water treatment and a judicious choice of the technology needs to be made by taking into consideration the required level of treatment and the life cycle cost.

Aerobic treatment schemes based on the established sequence of screening, grit removal, activated-sludge-process and its variants, anaerobic sludge digester and sludge drying beds, chlorination is quite suitable for large cities where either land is not easily available or it is too expensive.

It is also to be noted that anaerobic schemes like Upflow Anaerobic Sludge Blanket (UASB) reactor, Anaerobic Baffled Reactor (ABR) followed by waste stabilization ponds and constructed wetlands entail lower O & M costs and are likely to be a better choice in Medium and Small Towns based on techno-economic analysis. Trickling Filter is also considered a low cost aerobic process.

For the purpose of present study, the following nine technology options (combinations) are investigated for their suitability in STPs of capacity 1 MLD, 2 MLD and 5 MLD. In order to account for possibility of high land costs in SBM-U 2.0 Cities, technologies like SBR, EA and MBBR are also included in the techno-economic analysis.

2.2 Used Water (Influent) Parameters

The design of unit processes in a Used Water treatment plant requires inputs in the form of influent characteristics, site location, temperature, altitude etc. The design is carried out for a city in central India with a population of less than 20000 as per census 2011. Table 2.1 presents the design data used.

Table 2. 1 Design input parameters considered for the present study

S. No.	Design input parameter	Value
1	Latitude (City in central India)	21.14°N
2	Elevation	310 m
3	Lowest average December / January temperature	14.5° C
4	Highest average May / June temperature	29.5°C
5	Per capita water supply	135 lpcd
6	Peak factor	3.00
7	Influent BOD	250 mg/l
8	Influent COD	425 mg/l
9	Influent TKN	50 mg/l
10	Influent TSS	250 mg/l

Note: *For small and medium towns, the design input paramters should be determined by collecting sample(s) from the outfall drains, especially for non-mechanized technologies such as WSP, ABR, CWL, and UASB for designing the various unit processes.*

2.3 Design of Primary Treatment Processes

In the present study, the common primary treatment components considered in the design of the Used Water treatment plants are presented in Table 2.2.

Table 2. 2 Project components and their applicability to different technologies

S. No.	Component	Technology in which provided
1	Wet well with pumping	All
2	Coarse Screens	All
3	Fine Screens	All
4	Grit Chamber	All

Apart from the above common elements in primary treatment, the following secondary treatment components are also designed as common elements:

i. Secondary settling tank for activated sludge processes (ASP, EA and MBBR)

ii. Secondary settling tank for extended aeration process

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iii. Chlorination contact tank for all technologies

2.3.1 Design of 2 MLD Wet Well with Pumping Machinery

The wet well is the first component of any treatment plant and it is the chamber in which the sewer outfall empties. Usually it is circular, but rectangular wet wells are also employed. The depth of the wet well depends on the depth of the outfall sewer. In the present study, the depth of invert of outfall sewer is assumed as 3 m. Similarly, the lift above the ground level is taken as 4.4 m and it represents the level at which the flow is discharged into the screen chamber. Submersible pump is employed for lifting the influent from the sump to the primary treatment chamber. The general layout of the proposed wet well arrangement is presented in Figure 2.1.

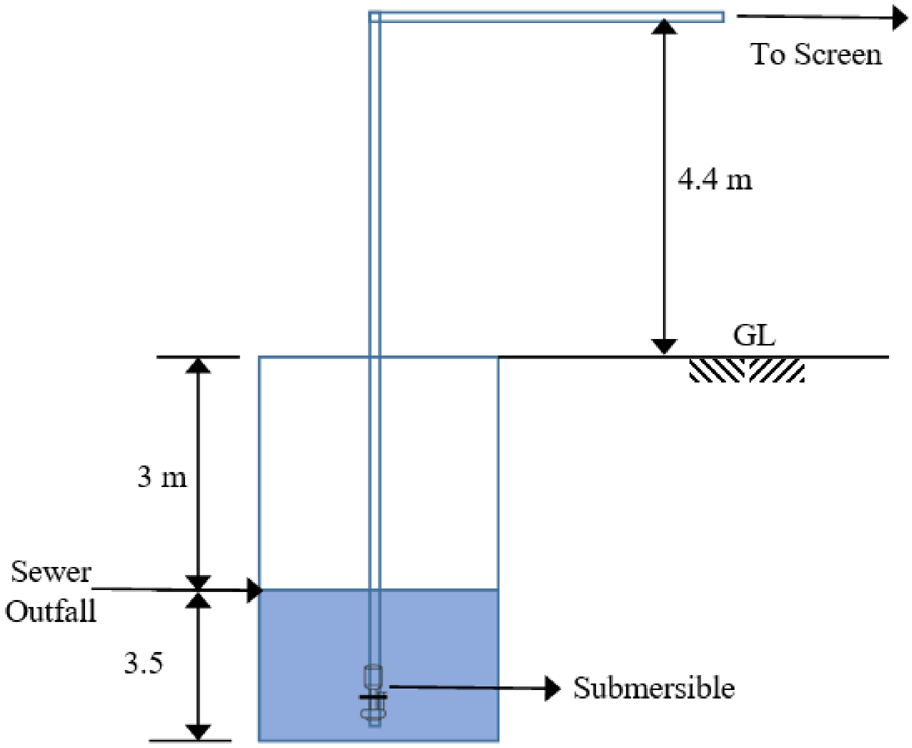

Figure 2. 1 Conceptual representation of the wet well with submersible pump

Note: *In case of WSP, ABR, CWL and UASB pumping head of 4.4 m may not be required.*

The detailed design of 2 MLD wet well is presented in Table 2.3. Circular wet wells are designed for all the three capacities, i.e. 1 MLD, 2 MLD and 5 MLD wet wells. The designs are based on practically valid assumptions for diameter of the outfall pipe, suction head, delivery head, minor losses, efficiency of the submersible pump set etc.

Table 2. 3 Design calculations of 2 MLD wet well with submersible pump

S. No.	Item	Formula	Quantity	Unit
1	Average flow	Capacity of the treatment plant	2	MLD
2	Average flow	MLD x 10^6/1000	2000	m^3/day
3	Average flow	m^3/day /(24 x 60 x 60)	0.023	m^3/s
4	Average flow	m^3/s x 60	1.39	m^3/minute
5	Peak factor	For a population up to 20, 000 as per CPHEEO Manual	3	
6	Peak flow	average flow x peak factor	4.17	m^3/minute
7	Volume of wet well	V = TQ/4 T = 15 minutes for small pumps (Reference: CPHEEO Manual)	15.62	m^3
8	Minimum depth	Below invert of sewer	2.0	m
9	Required Area of the well	Volume / depth	7.81	m^2
	Provide a circular wet well			
10	Computed well diameter	$d = \sqrt{\frac{4 \ x \ Area}{\pi}}$	3.15	m
11	Adopted well diameter		3.50	m
	Obtain Pump Capacity			
12	Diameter of invert pipe	Assumed	300	mm
13	Static head	H = suction head + Delivery head	10	m

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S. No.	Item	Formula	Quantity	Unit
14	Minor losses	10% of Manometric head	1	m
15	Total manometric head	Suction Head + Delivery Head + Minor losses	11	m
16	Efficiency	65%	0.65	
17	Pump Capacity	$P = \gamma Q H / \eta$	3.84	KW
18	Provided pump capacity	Provide 2 KW x 2 working 2 KW x 1 Standby	4	KW

Green cells: Input variable data to be provided Yellow cells: standard data from CPHEEO manual or other standards Red Cells: Design output

2.3.1.1 Design steps of 2 MLD Wet Well

1. Average flow = 2 MLD
2. Average flow = 2 x 10^6 / 1000 = 2000 m^3 / day
3. Average flow = 2000 / (24 x 60 x 60) = 0.023 m^3/s
4. Average flow = 0.023148 x 60 = 1.39 m^3/min
5. Peak factor = 3.00

Peak factor: The present study is undertaken for a city with a latitude of 21.14^o N and is therefore one of the centrally located cities in India. The population of the city is taken as 20, 000 as per Census 2011. As per Table 3.2 of CPHEEO Manual, the peak factor for the city is 3.00. For other cities suitable peak factor can be taken based upon the table reproduced below for ready reference. The wet well is designed for peak flow.

For a population of 20, 000 the sewage generation is obtained as follows:
= 20, 000 x 135 x 0.8 / 10^6 = 2.16 MLD. Hence for design of a 2 MLD STP, the population is taken as 20, 000 and the peak factor 3.0

Table 3.2 from CPHEEO Manual

Table 3.2 Peak factor for Contributory Population

Contributory Population	Peak Factor
up to 20,000	3.00
Above 20,001 to 50,000	2.50
Above 50,001 to 7,50,000	2.25
above 7,50,001	2.00

Source: CPHEEO, 1993

6. Peak flow = 1.39 x 3 = 4.17 m^3/min

7. **Volume of wet well:** The volume of wet well is calculated based upon the formula V = T x Q / 4 furnished in section 4.6.6 of the CPHEEO manual.

   Where:

   V = Effective volume of wet well (in cubic meters)
   T = Time for one pump cycle (in minutes) = 15 minutes.
   Q = Pumping rate (cubic meters per minute)

   V = 15 x 4.17 / 4 = 15.62 m^3

8. **Minimum depth below invert of sewer:** The depth of the wet well required is governed by Table 4.1 in 4.6.6 of CPHEEO Manual. This is governed by the height of the submersible pump set and the floor clearance. In the present study the depth below the invert of the sewer is taken as 2 m.

9. Required area of the well = Volume / depth = 15.62 / 2 = 7.81 m^2

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10. Well diameter = $\sqrt{4 * 7.81 / \pi}$ = 3.15 m

11. Adopted well diameter (rounded off) = 3.50 m

    The minimum diameter of the well is adopted as 3.00 in order to accommodate maximum of three submersible pumps, including standby pump.

12. Assume diameter of the invert pipe = 300 mm

13. **Static Head:** The static head is defined as the sum of suction head and delivery head. For a centrifugal pump it represents the vertical distance from the water level in the sump to the maximum height water is lifted. In case of a submersible pump it is taken as the level from the centre of the submersible pump to the maximum height up to which water is lifted. The concept of static head in terms of suction and delivery heads is presented in the accompanying sketch. Figure 2.2 illustrates the static head concept.

14. **Minor Losses:** The minor head losses are taken as 10% of the static head. If an exact estimate of head loss is available, the same can be used.

15. **Total Manometric Head:** The manometric head is the sum of static head and losses.

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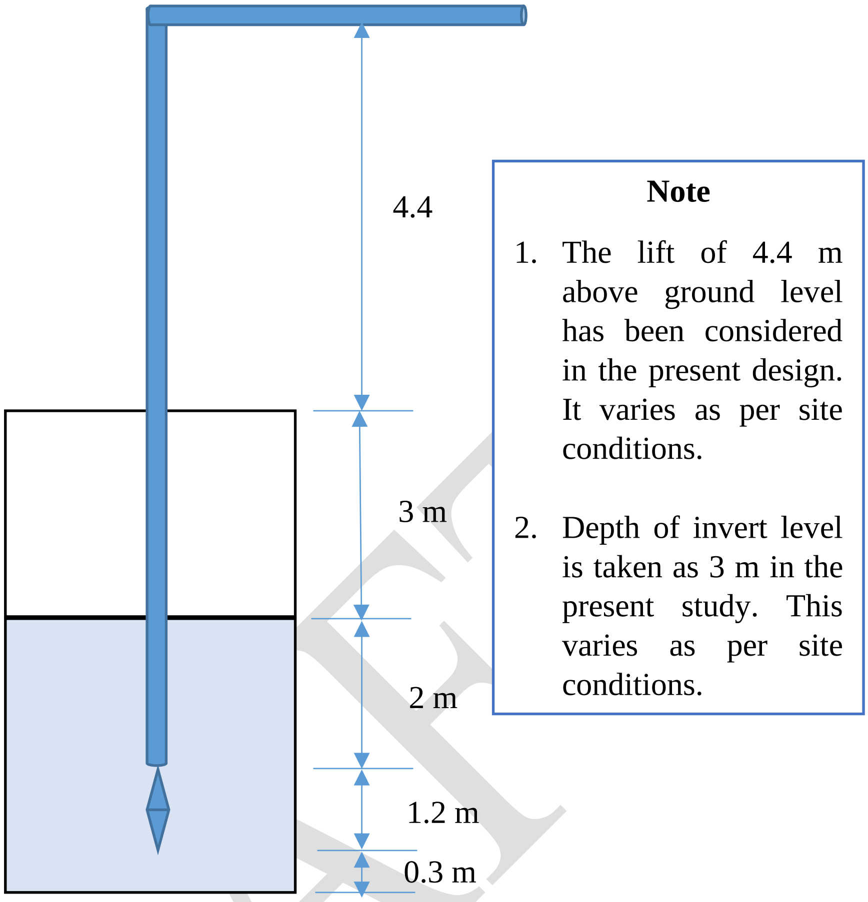

Figure 2. 2 Breakup of the static head of 10 m

In the present design the depth of invert of outfall sewer is taken as 3 m below GL, the depth of sewage below the invert is taken as 2 m, the pump height is taken as 1.2 m and the bottom clearance is taken as 0.3 m. The lift up to the coarse screen chamber is taken as 3.8 m. The various heads are shown in the accompanying sketch.

Static Head = Suction head + delivery head = vertical distance from the centre of the submersible pump to the maximum lift = (1.2/2) + 2 + 3 + 4.4 = 10 m

Head losses = 10% of static head = 0.1 x 10 = 1 m

Manometric Head = 10 + 1 = 11 m.

16. **Overall Efficiency:** From Table 4.3 of CPHEEO Manual, the overall efficiency for submersible pump is 65%. The efficiency is maximum for horizontally mounted centrifugal pump. Appropriate value may be taken from the table.

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Table 4.3 from CPHEEO Manual

Table 4.3 Efficiencies of pumps to be adopted for design purposes

No.	Type of Pump Set	Efficiency
1	Horizontal foot mounted centrifugal pump sets	0.85
2	Vertical shaft centrifugal pump sets	0.8
3	Submersible pump sets	0.65
4	Positive displacement pump sets	0.40

Note: *It is suggested that appropriate value of efficiency may be taken as suggested in the pump manufacturer manual.*

17. **Pump capacity:** The capacity of the hydraulic pump is obtained for the dry weather flow as per 4.5.4 of CPHEEO manual. The general practice is to provide 3 pumps for a small capacity pumping station comprising of (a) 1 pump of 1 DWF, (b) 1 of 2 DWF and (c) 1 of 3 DWF capacity. Alternatively, the number of pumps can also be chosen to be in multiples of DWF flow and provide a 100% standby capacity for peak flow.

Specific weight of sewage: Specific gravity of sewage is not mentioned in the CPHEEO manual. In literature it is mentioned as slightly more than 1, say 1.2. In the present calculation the specific gravity of sewage is taken as 1.

Hence Specific Weight = γ = 9810 N / m^3

$P = \gamma\ Q\ H_m / \eta$ Watts

P = 9810 x 0.023148 x 11 / 0.65 = 3.8429 KW

18. Provided pump capacity, P = say 4 KW.

Provide 3 Nos. 2 KW submersible pumps, 2 Working + 1 Standby thereby ensuring 50% standby capacity for peak flow as per the clause of CPHEEO manual.

Flow Equalization: As per 5.6.3, STPs are designed for DWF. When the peak factor exceed 3 (for small towns with population less than 20, 000) by a wide margin, it is advisable to equalize the sewage before feeding to the STP units.

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Similar calculations are carried out for 1 MLD and 5 MLD wet wells. The summary of the outcome is presented in Table 2.4.

Table 2. 4 Summary of the wet well design and submersible pump sets

S. No.	Item	Quantity	Size	Unit
	1 MLD			
1	Diameter		2.5	m
2	Depth below GL		6.5	m
3	Submersible pumps	3 Nos. 2 W + 1 S	1 KW each	KW
	2 MLD			
1	Diameter		3.5	m
2	Depth below GL		6.5	m
3	Submersible pumps	3 Nos. 2 W + 1 S	2 KW each	KW
	5 MLD			
1	Diameter		4.5	m
2	Depth below GL		6.5	m
3	Submersible pumps	3 Nos. 2 W + 1 S	5 KW each	KW

2.3.2 Design of 2 MLD Coarse Screen, Medium Screen and Grit Chambers

Design of coarse screen, medium screen and grit chamber is carried out as per the specifications contained in the CPHEEO’s ‘Manual on Sewerage and Sewage Treatment Systems – 2013’. The output for 2 MLD plant is presented in Table 2.5, Table 2.6 and Table 2.7 respectively. The summary of the designs for 1 MLD, 2 MLD and 5 MLD plants is presented in Table 2.8.

Table 2. 5 Design calculations summary for the coarse Screen for 2 MLD STP

1	Average Flow	2.00	MLD
2	Average Flow	0.02315	m^3/s

3	Peak Flow :	Peak Factor	3	6.00	MLD
4	Peak Flow :			0.069	m^3/s
5	Minimum Flow	Factor	2	1.00	MLD
6	Minimum Flow			0.01157	m^3/s
	DESIGN OF COARSE SCREEN				
7	Peak Flow			0.069	m^3/sec
8	Coarse Screen Opening			25.00	mm
9	Depth of water in screen			0.30	m
10	velocity through screen			0.70	m/s
11	Area of screen			0.099	m^2
12	Angle of inclination with the horizontal			45.00	degree
13	Sin (Angle of Inclination)			0.707	
14	Vertical area of the screen			0.070	m^2
15	Free Board			0.30	m
16	Length of screen			0.85	m
17	Vertical width of opening			0.23	m
18	Inclined width of opening			0.33	m
19	No. of openings			10.00	Nos.
20	Number of bars			9.00	Nos.
21	Let width of each bar be			10.00	mm
22	Total width of screen			340.00	mm

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23	Let width of each side wall be	50.00	mm
24	Total width of channel: Provide	440.00	mm
25	Size of Coarse Screen channel - Length	2939.24	mm
26	Provided coarse screen channel length	3.00	m
27	Approach velocity in Channel u/s of Screen	0.68	m/s
28	Velocity through Screen	0.92	m/s
29	Head Loss No Clogging	0.028	m
30	Velocity when 50% clogging	1.85	m/s
31	Head Loss when 50% clogging	0.22	m

Green cells: Input variable data to be provided   Yellow cells: standard data from CPHEEO manual or other standards   Red Cells: Design output

2.3.2.1 Design Steps of Coarse Screen

1. Average flow = 2 MLD

2. Average flow = 2 x 10^6/(1000 x 24 x 60 x 60) = 0.02315 m^3/s

3. Peak factor = 3.00

   The present design is undertaken for a city with a population < 20, 000 as per census 2011. As per clause 3.5 of CPHEEO manual the peak factor for this population is 3.00. The peak factor for different populations is given in Table 3.2 of the CPHEEO Manual.

Table 3.2 from CPHEEO Manual

Table 3.2 Peak factor for Contributory Population

Contributory Population	Peak Factor
up to 20,000	3.00
Above 20,001 to 50,000	2.50
Above 50,001 to 7,50,000	2.25
above 7,50,001	2.00

Source: CPHEEO, 1993

   Peak flow = 3.00 x 2 = 6.0 MLD

4. Peak flow = 0.023148 x 3 = 0.06944 m^3/s

5. Minimum flow factor = 0.5

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As per clause 3.5 of CPHEEO manual, 'the minimum flow may vary from 1/3 to 1/2 of average flow'.

6. Minimum flow = 0.02315 x 0.5 = 0.01157 m^3/s

7. Peak flow for coarse screen design = 0.06944 m^3/s

8. Coarse screen opening = 25 mm

As per clause 5.6.1.1 of CPHEEO Manual, 'It is usual to provide a bar screen with relatively large openings of 25 mm'.

9. Depth of water in the screen = 300 mm (Initial assumption)

The trail depth may have to be adjusted if the resultant velocity and head loss do not correspond to the limits specified in CPHEEO manual.

10. Velocity through the screen = 0.7 m/s (Initial assumption)

As per clause 5.6.1.8 of CPHEEO manual, 'Velocities of 0.6 to 1.2 m/s through the open area for the peak flows have been used satisfactorily'. Higher trail velocity may be used if the resultant velocity and head loss do not correspond to the limits specified in CPHEEO manual.

11. Area of the screen = Peak flow / velocity through the screen

= 0.06944 / 0.7 = 0.09921 m^2

12. Angle of inclination with the horizontal = 45^o

As per clause 5.6.1.1 of CPHEEO Manual, '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.'

In the present study, manual racks are assumed with an angle of 45^o. In other designs the angle may be varied up to 60^o or mechanical racks may be used, which are generally erected almost vertically.

13. Sin (45) = 0.70739

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14. Vertical area of the screen = Area x Sin 45 = 0.09921 x 0.70739 = 0.07018 m^2.

15. Free Board = 0.3 m Assumed. No specific value is specified in the CPHEEO manual.

16. Length of the screen = (Depth of water in screen + Free board) / sin(θ)
= (0.3 + 0.3) / 0.70739
= 0.84819 m

17. Vertical width of opening = 0.07018 / 0.3 = Vertical area / depth = 0.23392 m

18. Inclined width of the opening = 0.09921 / 0.3 = inclined area / depth = 0.33069 m

19. No. of openings = 0.23392 x 1000 / 25 = 9.3568, say 10 Nos.

20. No. of bars = 10 – 1 = 9 Nos.

21. Width of each bar = 10 mm
Clause 5.6.1.1 on coarse screens does not specify bar width. However, As per clause 5.6.1.2 of CPHEEO manual, '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.' Further in APPENDIX A 5.6, Assumed width of rectangular bars = 10 mm.

22. Width of the screen = 10 x 25 + 9 x 10 = 340.00 mm

23. Assume the width of each side wall of the screen = 50 mm. (Assumed)
This depends upon the material of construction and the structural design of the chamber.

24. Total width of the screen chamber = 340 + 2 x 50 = 440 mm, say 450 mm.

25. Length of the screen chamber = 5B + H cotθ + 5B
= 5 x 0.23392 + (0.3 + 0.3) x cot 45 + 5 x 0.23392
= 2.9392 m

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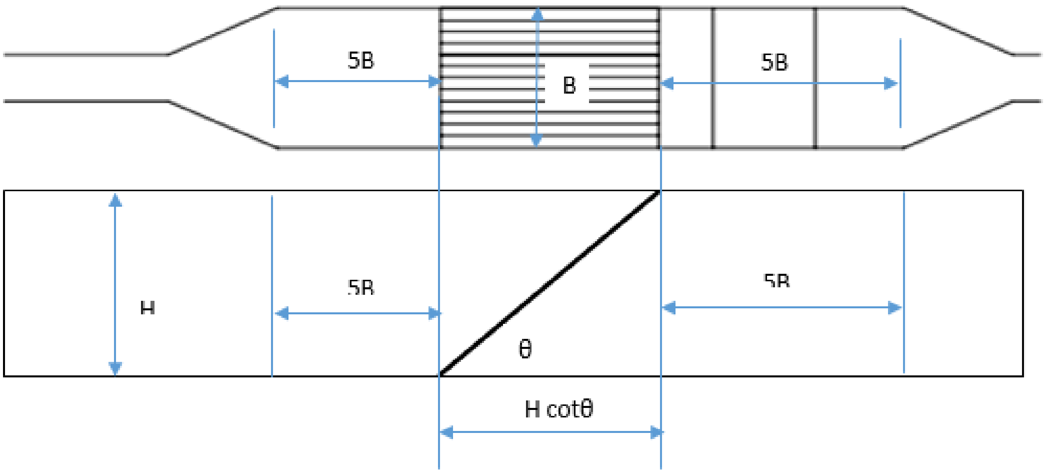

Figure 2. 3 Elements of a coarse screen and chamber

Coarse screen chamber length is computed based upon clause 5.6.1.8 of CPHEEO manual, which states, '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 screen is ideal for good hydraulics'. While it is mandatory to provide a straight channel of length 5B before the screen, a similar length is preferable after the screen. Figure 2.3 shows the typical layout of the coarse screen and its chamber.

26. Provided length of the coarse screen chamber = 3.0 m

27. Approach velocity in the channel on the u/s side of the screen = flow / approach area

= 0.06944 / (0.34 x 0.3) = 0.68082 m/s

This is greater than 0.3 m/s. Hence OK.

As per clause 5.6.1.8 of CPHEEO manual, 'Further, the velocity at low flows in the approach channel should not be less than 0.3 m/s to avoid deposition of solids'.

28. Velocity through the screen = flow / screen area

= 0.05208 / (0.3 x 0.025 x 7) = 0.92593 m/s

The velocity through the screen lies between 0.6 to 1.2 m and hence OK.

As per clause 5.6.1.8 of CPHEEO manual, 'Velocities of 0.6 to 1.2 m/s through the open area for the peak flows have been used satisfactorily'.

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29. As per clause 5.6.1.9 of the CPHEEO manual, '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:

$h = 0.0729(V^2 – v^2)$ … Equation 5.1 of the CPHEEO manual

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.

Head loss without clogging, $h = 0.0729 \times (0.92593^2 – 0.68083^2) = 0.02871$ m

The head loss is less than the permitted value of 0.15 m for non-clogged condition. Hence OK.

30. Velocity for 50% clogged screen = 2 x velocity without clogging
= 2 x 0.92593 = 1.85185 m/s.

31. Head loss with 50% clogging, $h = 0.0729 \times (1.85185^2 – 0.68083^2) = 0.2162$ m

As per clause 5.6.1.9 of CPHEEO manual, the maximum loss with clogged condition is 0.3 m. Since the present loss of 0.2162 m < 0.3 m, the design is OK.

Table 2. 6 Design calculations summary for the medium Screen for 2 MLD STP

	DESIGN OF MEDIUM SCREEN		
1	Peak Flow	0.069	m^3/s
2	Velocity through screen, Assume	0.80	m/s
3	Area	0.087	m^2
4	Depth of flow taken	0.40	m
5	Hence width of opening	0.22	m

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6	Clear opening between adjacent bars of screen	12.00	mm
7	Bars thickness of screen	10.00	mm
8	No of Openings	19.00	No.
9	No of Bars	18.00	No.
10	Width of the screen	408.00	mm
11	Angle of inclination with the horizontal	75.00	degree
12	Taking width of screen	400.00	mm
13	Nos of opening will be	19.00	Nos.
14	Nos of bars(10 mm thickness)	18.00	Nos.
15	Free Board	0.30	m
16	Sin (Angle of Inclination with horizontal)	0.97	
17	Inclined length of Screen	0.72	m
18	Length of medium screen chamber	4187.56	m
19	Chamber length adopted for medium screen	4.50	m
20	Width of each side wall	50.00	mm
21	Total width of channel	0.50	m
22	Velocity in Channel u/s of Screen	0.43	m/s
23	Velocity through Screen	0.76	m/s
24	Head Loss through screen	0.028	m
25	Velocity through screen when 50% clogged	1.52	m/s
26	Head Loss when 50% clogging	0.15	m/s

Green cells: Input variable data to be provided Yellow cells: standard data from CPHEEO manual or other standards Red Cells: Design output

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2.3.2.2 Design Steps of Medium Screen

1. Peak flow = 0.06944 m^3/s

2. Velocity through the screen = 0.8 m/s (Initial assumption)

   As per clause 5.6.1.8 of CPHEEO manual, 'Velocities of 0.6 to 1.2 m/s through the open area for the peak flows have been used satisfactorily'. Higher trail velocity may be used if the resultant velocity and head loss do not correspond to the limits specified in CPHEEO manual.

3. Area of the screen required = peak flow / velocity

   = 0.06944 / 0.8 = 0.08681 m^2

4. Depth of water in the screen = 400 mm (Initial assumption)

   The trail depth may have to be adjusted if the resultant velocity and head loss do not correspond to the limits specified in CPHEEO manual.

5. Width of the opening = Area / depth = 0.08681 / 0.4 = 0.21701 m

6. Clear opening between adjacent bars of the screen = 12.00 mm

7. Bar thickness of screen = 10.00 mm

   As per clause 5.6.1.2 of CPHEEO manual, 'Medium bar screens have clear openings of 12 mm. 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.'

8. No. of openings in the screen = 0.217 x 1000 / 12 = 18.08, say 19.

9. No. of bars = 19 – 1 = 18

10. Width of the medium screen = 19 x 12 + 18 x 10 = 408 mm

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11. Angle of inclination of the screen with horizontal = 75^o

    As per clauses 5.6.1.1 and 5.6.1.2, the medium screens are mechanically raked units and such racks are generally erected almost vertically. Hence the angle of 75^o is OK.

12. Adopted width of the screen = 400 mm.

13. Recalculate the No. of opening based on the adopted width of the screen.

    No. of openings = (400 + 10) / (12 + 10) = 18.6364, say 19.

    Let there be ‘n’ openings of 12 mm each. There will be (n-1) bars of 10 mm thickness.

    12 n + (n-1)10 = 400

    12n + 10n – 10 = 400

    (400 + 10) = (12+ 10) n

    n = (400 +10) / (12+10) = 18.6364, say 19 Nos.

14. No. of 10 mm thick bars = 19 - 1 = 18

15. Free board = 300 mm

    No specific value is specified in the CPHEEO manual for free board in screen chamber.

16. Sine (75) = 0.96592

17. Inclined length of the screen = (0.4+0.3) / 0.96592 = 0.72467 m

18. Length of chamber = 10 x 400 +(0.4+0.3)*1000*cot(75) = 4187.5644 mm

19. Provide a chamber of length = 4.5 m

20. Let width of each side wall be = 50 mm

21. Total width of the medium screen chamber = 400 + 2 x 50 = 500 mm = 0.5 m

22. Velocity in the approach channel (u/s of the screen) = 0.06944 / (0.4 x 0.4) = 0.434 m/s

    This is greater than 0.3 m/s. As per clause 5.6.1.8 of CPHEEO manual, this is OK.

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23. Velocity through the clean screen = 0.06944 / (0.4 x 19 x 12/1000) = 0.76145 m/s

24. Head loss through the screen, h = 0.0729 x (0.76145^2 – 0.434^2) = 0.02854 m

25. Velocity through 50% clogged screen = 2 x 0.76145 = 1.52290 m/s

26. Head loss under 50% clogging = 0.0729 x (1.5229^2 – 0.434^2) = 0.15534 m

Table 2.7 Design calculations Summary for the Grit Chamber for 2 MLD STP

	DESIGN OF GRIT CHAMBER		
1	Basic formula for settlement of grit	$\frac{Q}{A} = \frac{nV_S}{(1-\eta)^{-n}-1}$	
2	Here η value taken	75.00	%
3	Say	0.75	
4	Here V_S value taken	1528.97	$m^3/m^2/day$
5	Here 'n' value taken-1/8 (for very good performance)	0.12	
6	Hence surface over flow rate (Q/A)=(Vs*n)/(((1-η)^-n)-1)	1010.11	$m^3/m^2/day$
7	Say	1010.00	$m^3/m^2/day$
8	Peak flow	6000	(m^3/day)
9	No. of grit chambers proposed	2	Nos.
10	Peak flow per chamber	3000.00	(m^3/day)
11	Required for peak flow	2.97	m^2
12	Detention time	1	minute
13	Selected width of the tank	0.6	m
14	Selected depth of the tank	0.3	m
15	Particle size for critical displacement velocity	0.15	mm

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16	Critical displacement velocity	0.197	m/s
17	Horizontal velocity in the chamber	0.19	m/s
18	Free board	0.3	m
19	Provision of space for grit	0.3	m
20	Length of the tank	11.57	m
21	Adopted length of the tank	12.00	m
22	Total depth of the tank	0.90	m
23	This gives detention time as	62.21	Seconds

Green cells: Input variable data to be provided Yellow cells: standard data from CPHEEO manual or other standards Red Cells: Design output

2.3.2.3 Design Steps of Grit Chamber

1. Basic formula for settlement of grit is based on Stoke's law as described in sub-clauses of clause 5.6.2.7.1 of the CPHEEO manual.

   The practical equation for determination of surface overflow rate is given by equation 5.11 in clause 5.6.2.7.1.2 of the CPHEEO manual, which is stated as, 'Following equation could be used to determine the SOR for a real basin for a given efficiency of grit removal and basin performance.'

$$\eta = 1 - \left[1 + \frac{nV_s}{\frac{Q}{A}}\right]^{-\frac{1}{n}}$$ Equation 5.11of CPHEEO manual

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

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Equation 5.11 can be algebraically transformed as follows for expressing Q/A in terms of other variables.

$$\frac{Q}{A} = \frac{nV_s}{(1-\eta)^{-n} - 1}$$

2. Take efficiency, η = 75%

3. Efficiency expressed as fraction = 0.75

4. Take V_s = 1528.96702 $m^3/m^2/day$

Table 5.6 of the CPHEEO manual 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 (5.7) which is stated as:

$V_s = [0.707(S_s - 1)d^{1.6}\nu^{-1.6}]^{0.714}$ Equation 5.7 of the CPHEEO manual

The corresponding surface overflow rate for 100% removal of the particles based on equation 5.7 of CPHEEO manual is also given in Table 5.6 of the CPHEEO manual.

Table 5.6 from CPHEEO Manual

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

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

Equation 5.7 is used in design of grit chambers which are designed to remove particles of size 0.15 mm or 0.2 mm.

As per clause 5.6.2.7.1.1 of the CPHEEO manual, '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.'

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For particle size of 0.15 mm, equation 5.7 gives:

$V_s = (0.707 \times (2.65 - 1) \times (0.15 \times 10^{-3})^{1.6} \times (1 \times 10^{-6})^{-1.6})^{0.714} = 0.0177$ m/s

= 0.0177 x 24 x 60 x 60 =1528.96702 m^3 / m^2 / day

= 1528.97 m^3 / m^2 / day

(V_s is obtained as 1555 m^3 / m^2 / day in Table 5.6 as V_s is taken as 0.018 m/s instead of 0.0177 m/s)

5. n = 1/8 for very good performance

Relevant references for the choice of efficiency, V_s, and n are given below.

As per clause 5.6.2.7.1.2. 'The value of n is 1/8 for very good performance of the grit chamber. The design surface overflow rate will be 66.67% of the settling velocity of the grit particles to be removed to achieve 75% removal efficiency in grit chamber.'

6. Practical surface overflow rate is obtained by using equation 5.11 of the CPHEEO manual.

$$\frac{Q}{A} = \frac{nV_s}{(1-\eta)^{-n}-1} = 0.125 \times 0.0177 / ((1-0.75)^{-0.125}-1) = 0.01169 \text{ m/s}$$

= 0.01169 x 24 x 60 x 60 = 1010.1146 m^3 / m^2 / day

7. Adopted value of surface overflow rate = 1010 m^3 / m^2 / day

8. Peak flow = 6.0 MLD = 6000 m^3 / day

9. No. of girt chambers proposed = 2

10. Peak flow per chamber = 6000 /2 = 3000 m^3 / day

11. Area required for peak flow = Peak flow / surface over flow rate = 3000 / 1010 = 2.97030 m^2

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12. Detention time = 1 minute

    As per clause 5.6.2.7.1.3 of the CPHEEO manual, 'The detention time should not exceed 60 seconds.'

13. Choose width of the tank = 0.6 m

14. Choose depth of flow in the tank = 0.3 m

15. Particle size considered for settling velocity / critical displacement velocity = 0.15 mm

    The bottom scour and flow through velocity is the limit beyond which particles of 0.15 mm, once settled, may be again placed in motion and reintroduced into the stream.

16. Critical displacement velocity

    $V_c = K_c[(S_s - 1)gd]^{0.5}$ Equation 5.12 of CPHEEO manual

    Where K_c = 3 to 4.5, usually 4.0

    $V_c = 4x\ [(2.65 – 1) \times 9.81x\ 0.15 \times 10^{-3}]^{0.5}$

    V_c = 0.1971 m/s

    As per clause 5.6.2.7.1.4 of CPHEEO manual the bottom scour and flow through velocity is given by equation 5.12 of the manual.

17. Horizontal velocity in the chamber = peak flow / cross sectional area

    = (3000 / 24 x 60 x 60) / (0.6 x 0.3) = 0.1929 m/s

    The horizontal velocity is less than the critical displacement velocity. Hence OK.

18. Free board = 0.3. (Assumed). The CPHEEO manual does not specify free board for grit chamber. However, in Appendix A 5.7, in the design of grit chamber, a free board of 250 mm is used.

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19. Provision of space for grit = 0.3 m

    In Appendix A 5.7, in the design of grit chamber, grit storage space of 250 mm is used.

20. Length of the tank = horizontal velocity x 60 seconds = 0.1929 x 60 = 11.574 m

21. Adopted length of the grit chamber = 12 m

22. Total depth of the tank = depth + free board + grit space = 0.3 + 0.3 + 0.3 = 0.9 m

23. Detention time = Volume of chamber / flow = 0.6 x 0.3 x 12 / 0.034722 = 62.20 seconds

    The detention time is slightly more than the permitted value of 60 seconds. Hence OK.

Table 2. 8 Summary of the coarse screen, medium screen and grit chamber designs

S. No.	Component	Particulars	Capacity		
			1 MLD	2 MLD	5 MLD
1	Coarse Screen Chamber	Length	2.0 m	3.0 m	6.5 m
2		Width	0.30 m	0.45 m	1.00 m
3		Depth	0.6 m	0.6 m	0.6 m
4		Screen size	0.85 m x 0.165 m	0.85 m x 0.34 m	0.85 m x 0.83 m
1	Medium Screen Chamber	Length	3.5 m	4.5 m	8.0 m
2		Width	0.4 m	0.5 m	0.9 m
3		Depth	0.6 m	0.7 m	1.00 m
4		Screen size	0.62 m x 0.3 m	0.62 m x 0.4 m	1.04 m x 0.8 m
1		Length	12.0 m	12.0 m	12.0 m

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S. No.	Component	Particulars	Capacity		
			1 MLD	2 MLD	5 MLD
2	Grit Chamber	Width	0.3 m	0.6 m	0.8 m
3		Depth	0.9 m	0.9 m	1.20 m

2.4 Design of Waste Stabilization Ponds

Waste stabilization ponds are large shallow basins in which raw sewage is treated entirely by natural processes involving both algae and bacteria. They are suitable for treatment of sewage of small communities and townships in temperate and tropical climates in view of their simplicity and satisfactory performance indicated by actual usage in different parts of the world. Their popularity is attributed to the cost-effectiveness, reliability and ease of operation for treating domestic and industrial Used Water. The cost effectiveness is owing to the non-requirement of power, except sunlight energy, and minimum supervision for daily operation consisting of simply cleaning the inlet and outlet works. .

2.4.1 Working of Waste Stabilization Ponds

The waste stabilization action in the pond is due to the combined activity of algae, bacteria, and zoo plankton. Algae in the presence of sunlight liberate free oxygen by photo-synthesis action. The oxygen so liberated is utilized by the bacteria which break the organic matter present in the waste. During the process of assimilation, ammonia, carbon dioxide and other substances are liberated which are synthesized by the algal cells. Thus a symbiotic cycle is established between the bacteria and algae in these ponds.

The design criteria for waste stabilization pond is presented in IS : 5611 – 1967 (Reaffirmed 2002) Code of Practice for Construction of Waste Stabilization Ponds (Facultative Type). As per the observation in this code of practice, Under prevailing climatic conditions, stabilization ponds in India have shown an average reduction ranging from 80 to 90 percent of biochemical oxygen demand (BOD) at a loading rate of 150 to 325 kg/ha/day. The performance of stabilization ponds is comparable to that of conventional sewage treatment plants such as

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trickling filters, activated sludge process, etc. with regard to BOD removal, and higher in the removal of pathogenic organisms.

2.4.2 Types of Ponds

The waste stabilization ponds are of three types:

i. **Anaerobic ponds:** Anaerobic ponds are commonly 2 – 5 m deep and receive wastewater with loading rates equivalent to more than 3000 kg/ha.day for a depth of 3 m. BOD removal is achieved by sedimentation of solids, and subsequent anaerobic digestion in the resulting sludge. A properly designed anaerobic pond will achieve about 40% removal of BOD at 10^{o}C, and more than 60% at 20^{o}C. A short Hydraulic Retention Time (HRT) of 1.5 to 3 days is usually employed.

ii. **Facultative ponds:** Facultative ponds are usually 1-2 m deep and receive either raw sewage or effluent from anaerobic ponds. The surface loading rate is usually 100-400 Kg/ ha.day. The upper layers consist of an aerobic zone and the deeper layers exhibit anaerobic activity. The algae which grows on the surface produces oxygen which is consumed by aerobic bacteria in the middle of the pond, thereby degrading the BOD. In the lower zones of the pond, anaerobic digestion takes place. This process requires 2 -3 weeks water retention time, against the 2 -3 days in anaerobic pond. About 70 – 95% of the BOD is removed during this process.

iii. **Aerobic ponds:** Also called as maturation ponds, these are placed last in the pond treatment system. They are very shallow, and generally occupy very large surface areas. Their main function is the reduction of pathogenic organisms. The processes by which the pathogens are removed are multiple, and include sedimentation, lack of food and nutrients, solar ultra-violet radiation, high temperatures and pH, natural predators, toxins and natural die-off.

A combination of anaerobic pond followed by facultative pond and maturation pond (aerobic pond) can be adopted to achieve the desired level of treatment. Maturation pond is not employed as chlorination is resorted to for destroying the pathogens

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2.4.3 Design Calculations of Anaerobic Pond + Facultative Pond

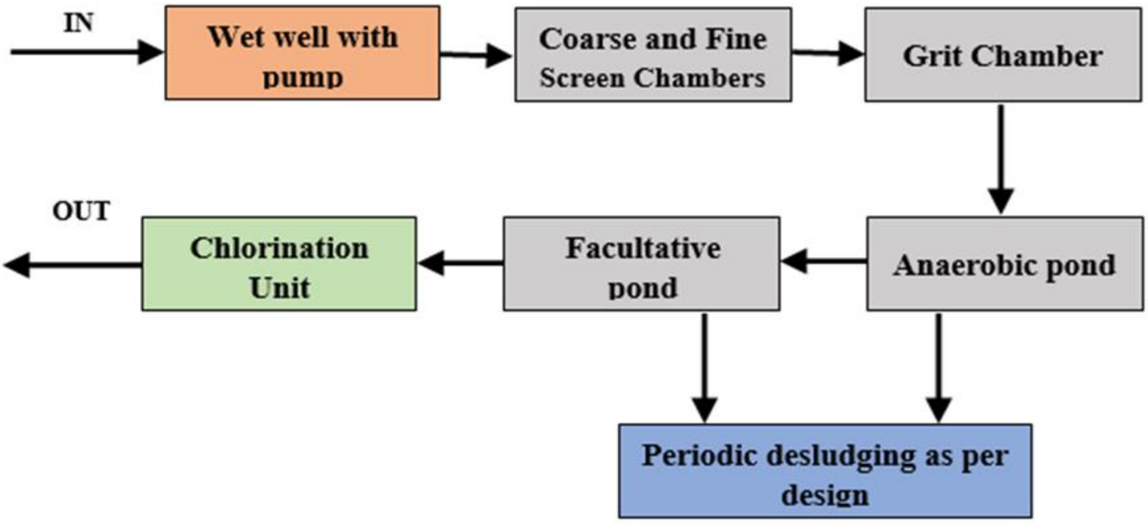

Figure 2.4 Conceptual representation of the Waste Stabilization Ponds

In the present study, the adopted scheme of treatment for the anaerobic pond – Facultative Pond combination is shown in Figure 2.4. The sludge is periodically emptied from the anaerobic and facultative ponds. Desludging of the anaerobic pond is done once in 2 to 5 years and the facultative pond every 5 -10 years. Maturation pond is not employed as chlorination is resorted to for destroying the pathogens

The design calculations of anaerobic pond of capacity 2 MLD is presented in Table 2.9 and the summary of 1 MLD, 2 MLD and 5 MLD is presented in Table 2.10. This is followed by the design of facultative ponds of capacity 1 MLD, 2 MLD and 5 MLD in Table 2.11. The design summary is presented in Table 2.13.

Table 2.9 design calculations of 2 MLD anaerobic pond

DESIGN OF 2 MLD ANAEROBIC POND					
S. No.	**Parameter**			**Value**	**Unit**
1	Average flow			2.00	MLD
2	Average flow			0.0232	(m^3/s)
3	Peak flow	Peak factor	3	6.00	MLD

4	Peak flow	0.069	(m^3/s)
DESIGN CALCULATIONS OF THE ANAEROBIC POND			
5	No of Anaerobic ponds to provide flexibility in O & M of ponds	1.00	Nos
6	Each handling flow of	2000.00	(m^3/day)
7	Say	0.023	(m^3/s)
8	Volume of anaerobic pond V=Li x Q /λ		
9	Li-Raw sewage BOD	250.00	mg/l
10	Q - average flow	2000.00	(m^3/day)
11	λ- volumetric BOD loading (Range between 100-400 gm/m3/day)		
12	λ=20T-100		
13	T=mean temperature in coldest month in ^{0}C	14.50	^{0}C
14	hence λ is	190.00	gm/m3/day
15	Hence Volume of tank(V)	2631.58	m^3
16	Detention time	1.32	day
17	Detention time of 2 days or more is desirable to achieve sufficient BOD removal	2.00	day
18	Hence Volume of each tank (V)	4000.00	m^3
19	Hence Volumetric BOD loading(λ) is	125.00	gm/m3/day
20	BOD removal efficiency(in %)=2T+20	49.00	%
21	For liquid depth	4.00	m

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22	Area at mid depth	1000.00	m^2
23	Length shall be L=2B		
24	Hence mid depth width $B=(area/2)^{1/2}$	22.36	m
25	Hence mid length L	44.72	m
26	Side slope of pond is 1V :2.0 H	2.00	m
27	Depth for Sludge	1.00	m
28	bottom below Mid depth	2.50	m
29	Free Board	1.00	m
30	Top of embankment above mid depth	3.50	m
31	Hence total top length(L)	58.72	m
32	Hence total top width(W)	36.36	m
33	Area at top	2135.15	m^2
34	Hence total bottom length (L)	34.72	m
35	Hence total bottom width(W)	12.36	m
36	Area at Bottom	429.18	m^2
ADOPTED SIZE OF ANAEROBIC POND			
37	No. of ponds	1	No.
38	Length (Top / Bottom / Mid depth)	59 / 35 / 45	m
39	Width (Bottom / Top / Mid depth)	36.5 / 12.5 / 22.5	m

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40	Depth (Including 1 m depth for sludge and 1.0 m for free board)	6.00	m

Green cells: Input variable data to be provided Yellow cells: standard data from CPHEEO manual or other standards Red Cells: Design output

2.4.3.1 Design Steps of 2 MLD Anaerobic Pond

REFERENCES FOLLOWED

A. 'Waste Stabilization Ponds and Constructed Wetlands Design Manual', S. Kayombo, T.S.A. Mbwette, J.H.Y. Katima, University of Dar es Salaam and N. Ladegaard, S.E. Jorgensen, Danish University of Pharmaceutical Sciences Section of Environmental Chemistry, Copenhagen, Denmark.

B. Decentralized Wastewater Treatment Systems (DEWATS) and Sanitation in Developing Countries, Editors: Andreas Ulrich, Stefan Reuter and Bernd Gutterer, Authors: Bernd Gutterer, Ludwig Sasse, Thilo Panzerbieter and Thorsten Reckerzugel, Published by Water, Engineering and Development Centre, Loughborough University, UK.

C. Wastewater Treatment Concepts and design Approach, G.L. Karia and R. A. Christian, PHI, 2015.

D. National Capital Region Planning Board Toolkit, available at the url: https://ncrpb.nic.in/NCRBP%20ADB-TA%207055/Sewerage/index.html

Input Data and Sewage Characteristics

1. Average Flow = 2 MLD. This is the capacity of the STP.

2. Average flow = 2 x 10^6 / (1000 x (24 x 60 x 60)) = 0.02315 m^3/s

3. Peak factor = 3

Table 3.2 from CPHEEO Manual

The present design is undertaken for a city with a population < 20, 000 as per census 2011. As per clause 3.5 of CPHEEO manual the peak factor for this population is 3.00. The peak factor for different populations is given in Table 3.2 of the CPHEEO Manual. The peak factor is however used for design of wet well, coarse screen, medium screen and grit chamber. The recommended hydraulic retention time of anaerobic pond is 2 days and hence the design of anaerobic pond is done for average flow since the peak flow is absorbed in the reactor without the necessity of additional capacity.

Table 3.2 Peak factor for Contributory Population

Contributory Population	Peak Factor
up to 20,000	3.00
Above 20,001 to 50,000	2.50
Above 50,001 to 7,50,000	2.25
above 7,50,001	2.00

Source: CPHEEO, 1993

Peak flow = 2 x 3 = 6 MLD

4. Peak flow = $6 \times 10^6 / (1000 \times (24 \times 60 \times 60)) = 0.06944\ m^3/s$

5. No. of anaerobic ponds to provide flexibility in O & M of pond = 1

In the present design a single pond is considered. The anaerobic ponds are generally used in tandem with facultative ponds. In such a case, the number of anaerobic ponds can be taken as same as the number of facultative ponds. This is purely from the point of view of operational ease.

6. Flow through each of the anaerobic ponds = 2000 m^3/day

7. Say = 0.02315 m^3/s

8. Volume of anaerobic pond is given by

$V = L_i Q / \lambda$

As per clause 2.6.3 of Reference A, 'The anaerobic ponds are designed on the basis of volumetric loading (λV, $g/m^3 . d$), which is given by:

$\lambda_v = L_i Q / V_a$ (2.12)

Where L_i is influent BOD (mg/l), Q is flow rate (m^3 /day), and V_a is anaerobic pond volume (m^3).

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9. L_i = Influent BOD = 250 mg/l

   As per clause 5.1.4.1 of CPHEEO Manual, '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. From Table 5.4, the recommended BOD is 250 mg/l.

10. Q (Average) = 2000 m^3/day. This is design data.

11. λ = Volumetric BOD loading.

    As per Table 2.7 of Reference A, the design value of volumetric loading is given as a function of temperature.

Table 2.7 from Reference A

Table 2.7 Design value of permissible volumetric BOD loadings on, and percentage BOD removal in, anaerobic ponds at various temperatures (from Mara and Pearson, 986; Mara *et al.* 1997)

Temperature (°C)	Volumetric loading (g/m^3.day)	BOD removal (%)
< 10	100	40
10 - 20	20T – 100	2T+20
20-25	10T+100	2T+20
>25	350	70

The value of λ ranges between 100 – 400 g / (m^3. day).

The precise value is obtained from Table 2.7 based on the temperature of the site.

12. The volumetric loading is given by 20T – 100 g / (m^3. day) for temperature range of 10-20 C.

13. In the present study, the plant is assumed to be located at a latitude of 21.14° N (Central India) where the lowest winter temperature is obtained as 14.5 C from the IMD records.

    T = 14.5° C

14. $\lambda = 20 \times 14.5 - 100 = 190$ g / (m^3 . day)

15. Volume of the tank = V = L_i Q / λ = 250 x 2000 / 190 = 2631.5789 g / (m^3 . day)

16. Detention time = Volume of tank / average flow = 2631.5789 / 2000 = 1.31579 days.

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As per clause 2.6.2 of Reference A, ‘A retention time less than one day should not be used for anaerobic ponds; if it occurs, however, a retention time of one day should be used, and the volume of the pond should be recalculated.’

17. Adopted detention time = 2 day.
In the present study the detention time of 2 days is used in order to achieve good BOD removal in the anaerobic phase so that the load on facultative pond is reduced.

18. Volume of the tank = 2 x 2000 = 4000 m^3

19. Resulting value of volumetric BOD loading = Li x computed detention time / adopted detention time = 190 x 1.31579 / 2 = 125 g / (m^3 . day)

20. BOD removal efficiency = (2T + 20)%

As per Table 2.7 of Reference A, the BOD removal (%) is (2T + 20)% for temperature range of 10 - 20^o C.

= 2 x 14.5 + 20 = 49%

21. Depth of the tank = 4 m (Assumed)
As per clause 2.3.1 of Reference A, ‘Anaerobic ponds are commonly 2 – 5 m deep’

22. Area of the tank at mid-depth = Volume / liquid depth = 4000 / 4 = 1000 m^3.

23. ‘Anaerobic ponds are slightly rectangular (sometime square) with typical length/breath (L/B) ratio of 1 to 3. (Reference: Wastewater stabilization ponds (WSP) - An ideal low cost solution for wastewater treatment around the world, Sheikh Mahabub Alam Department of Tourism and Hospitality Management, The Peoples University of Bangladesh, Dhaka, Bangladesh.

In the present study, adopt L = 2B

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24. Mid depth width B = $\sqrt{Area/2}$ = $\sqrt{1000/2}$ = 22.3607 m

25. Mid length L = 2B = 2 x 22.3607 = 44.7214 m

26. Side slope of pond = 1 V:2 H

AS per 6.2.3 of IS : 5611 – 1987 (reaffirmed 2002) – Indian Standard Code of Practice for construction of Waste Stabilization ponds (Facultative Type), 'Embankment slopes should not be steeper than: a) 2 to 2.5 horizontal to 1 vertical for unprotected earthen embankments, and b) 1.5 horizontal to 1 vertical for pitched or lined embankments.

27. Depth for sludge accumulation in the tank = 1 m (Assumed)

28. Bottom below mid depth = (4 + 1)/ 2 = 2.5 m

29. Free Board = 1 m (Assumed).
A higher value of free board is assumed in order to minimize the risk of overflowing.

30. Top of embankment above mid-depth = bottom below mid depth + Free board
= 2.5 + 1 = 3.5 m

31. Total top length of the pond (L) = 44.7 +2 x (3.5 x 2) = 58.72136 m

32. Total top width = 22.3607 + 2 x (3.5 x 2) = 36.36068 m

33. Area of the pond at the top = top length x top width = 58.72136 x 36.36068
= 2135.14855 m^2

34. Total bottom length = Mid length – (2 x bottom below mid depth x side slope)
= 44.7216 – (2 x 2.5 x 2) = 34.7216 m

35. Total bottom width = Mid depth width – (2 x bottom below mid depth x side slope)
= 22.36068 – (2 x 2.5 x 2) = 12.36068 m

36. Area at the bottom = bottom length x bottom width

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$$= 34.72136 \times 12.36068 = 429.17961 \text{ m}^2$$

The design summary presented in Table 2.10 lists the top length and width, bottom length and width and the mid-depth length and width. The depth of all the ponds (1, 2 and 5 MLD) is 6 m and the side slope is 1V : 1H. The adopted value confirm with the specification given in the BIS code.

Table 2. 9 Design summary of 1, 2 and 5 MLD anaerobic pond

STP capacity	Item	Size	Unit
1 MLD	Number of ponds	1	No.
	Top Length / Bottom Length / Mid-depth length	46 / 22 / 32	m
	Top Width / Bottom Width / mid-depth width	30 / 6 / 16	m
	Bund Side Slope	1 V : 2 H	-
	Depth (5 m + 1 m free board)	6.0	m
2 MLD	Number of ponds	1	No.
	Top Length / Bottom Length / Mid-depth length	59 / 35 / 45	m
	Top Width / Bottom Width / mid-depth width	36.5 / 12.5 / 22.5	m
	Bund Side Slope	1 V : 2 H	-
	Depth (5 m + 1 m free board)	6.0	m
5 MLD	Number of ponds	2	No.
	Top Length / Bottom Length / Mid-depth length	64 / 40 / 50	m
	Top Width / Bottom Width / mid-depth width	39 / 15 / 25	m
	Bund Side Slope	1 V : 2 H	-
	Depth (5 m + 1 m free board)	6.0	m

Table 2. 10 Design calculations of facultative pond

DESIGN OF FACULTATIVE WASTE STABILIZATION POND OF 1, 2 AND 5 MLD CAPACITY

S. No.	Design parameter	Design output

1	Capacity of the STP	1	MLD		2	MLD		5	MLD
2	Capacity of the STP	1000	m^3/day		2000	m^3/day		5000	m^3/day
3	Capacity of the STP	0.0116	m^3/s		0.0231	m^3/s		0.0579	m^3/s
4	Per capita supply	135	LPCD		135	LPCD		135	LPCD
5	Influent BOD_5 (As per Table 5.4 of CPHEEO Manual - 2013)	27	g/capita per day		27	g/capita per day		27	g/capita per day
6	Influent BOD_5 for Anearobic pond	250	mg/l		250	mg/l		250	mg/l
7	Influent BOD, for Facuiltative pond	127.5	mg/l		127.5	mg/l		127.5	mg/l
8	Effluent BOD_5 (As per Schedule VI of environment (protection) third Amendment Rules, 1993 - for Inland surface water)	30	mg/l		30	mg/l		30	mg/l
9	Latitude (Chosen for sample design corresponding to a centrally located city in the country.	21.14	degree		21.14	degree		21.14	degree
10	Average ambient temperature in December (https://nagpur.gov.in/geography-climate/)	14.5	ºC		14.5	ºC		14.5	ºC
11	Altitude of the city	310	m		310	m		310	m
12	Permissible pond loading rate according to latitude as per Table 1 of IS : 5611 - 1987 or Table 5.14 of CPHEEO manual	242.88	Kg/ha/day		242.88	Kg/ha/day		242.88	Kg/ha/day
13	Correction of pond loading rate for altitude = (1+0.003 x altitude in m)	1.93			1.93			1.93	

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14	Corrected pond loading rate as per latitude and elevation	125.84	Kg/ha/day		125.84	Kg/ha/day		125.84	Kg/ha/day
15	Pond loading rate according to temperature = 20T-120	170	Kg/ha/day		170	Kg/ha/day		170	Kg/ha/day
16	Adopt the average and the design pond loading rate	147.92	Kg/ha/day		147.92	Kg/ha/day		147.92	Kg/ha/day
17	Design pond loading rate, say	148	Kg/ha/day		148	Kg/ha/day		148	Kg/ha/day
18	BOD loading rate from the town	127.5	Kg/ha/day		255	Kg/ha/day		637.5	Kg/ha/day
19	Pond area required	0.86	ha		1.72	ha		4.31	ha
20	Depth of the pond	1.5	m		1.5	m		1.5	m
21	Pond volume	12922.30	m^3		25844.59	m^3		64611.49	m^3
22	Pond detention time	12.92	days		12.92	days		12.92	days
23	No. of ponds in parallel	2			2			2	
24	No. of ponds in series	1			1			1	
25	Total number of ponds	3			3			3	
26	BOD reaction rate K_1	0.2			0.2			0.2	
27	Percentage reduction of BOD required	0.76			0.76			0.76	
28	Detention time required for plug flow condition	7.23	days		7.23	days		7.23	days
29	Detention time required for completely mixed flow	10.94	days		10.62	days		10.94	days

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30	Population equivalent	7407			14815			37037	
31	Area per person (net)	1.16	m^2		1.16	m^2		1.16	m^2
32	Rate of sludge accumulation	0.07	m^3/person /day		0.07	m^3/person /day		0.07	m^3/person /day
33	Sludge volume in 1 year	518.52	m^3		1037.04	m^3		2592.59	m^3
34	Depth in the first cell to accommodate the sludge	2	m		2	m		2	m
35	No. of first cells allocated for sludge accumulation	2	No.		2	No.		2	No.
36	Depth of sludge in the first 2 cells	0.5	m		0.5	m		0.5	m
37	Volume of sludge in the first 2 cell(s)	2871.62	m^3		5743.24	m^3		14358.1 1	m^3
38	Duration of sludge cleaning	5.54	years		5.54	years		5.54	years
39	Area of each pond	0.29	ha		0.57	ha		1.44	ha
40	L/B ratio	4	dimensio nless		4	dimensio nless		4	dimensio nless
41	Length L	107.18	m		151.57	m		239.65	m
42	Breadth B	26.79	m		37.89	m		59.91	m
43	First 2 Tank L x B X D (Mid-Depth)	108 x 27 x 2	m		152 x 38 x 2.0	m		240 x 60 x 2	m
44	Subsequent Tanks L x B X D (Mid-Depth)	108 x 27 x 1.5	m		152 x 38 x 1.5	m		240 x 60 x 1.5	m
45	Length at mid-depth of primary and secondary tanks	108	m		152	m		240	m

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46	Width at mid-depth of primary and secondary tanks	27	m		38	m		60	m
49	Side slope of pond is 1V :2.0 H	2	m		2	m		2	m
50	Top of embankment above mid depth in primary tank	2	m		2	m		2	m
51	Tank Bottom below mid-depth in primary tank	1	m		1	m		1	m
52	Top of embankment above mid depth in secondary tank	1.75	m		1.75	m		1.75	m
53	Tank Bottom below mid-depth in Secondary tank	0.75	m		0.75	m		0.75	m
54	Top length in primary tank	116	m		160	m		248	m
55	Top width in primary tank	35	m		46	m		68	m
56	Bottom length in primary tank	104	m		148	m		236	m
57	Bottom width in primary tank	23	m		34	m		56	m
58	Top length in secondary tank	115	m		159	m		247	m
59	Top width in secondary tank	34	m		45	m		67	m
60	Bottom length in primary tank	105	m		149	m		237	m
61	Bottom width in secondary tank	24	m		35	m		57	m

Green cells: Input variable data to be provided Yellow cells: standard data from CPHEEO manual or other standards Red Cells: Design output

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2.4.3.2 Design Steps of 2 MLD Facultative Pond

REFERENCES FOLLOWED:

A. Wastewater Treatment for Pollution Control and Reuse, third edition, Soli J Arceivala and Shyam R Asolekar, Mc Graw Hill Education, 2007.

B. 'Waste Stabilization Ponds and Constructed Wetlands Design Manual', S. Kayombo, T.S.A. Mbwette, J.H.Y. Katima, University of Dar es Salaam and N. Ladegaard, S.E. Jorgensen, Danish University of Pharmaceutical Sciences Section of Environmental Chemistry, Copenhagen, Denmark.

C. Decentralized Wastewater Treatment Systems (DEWATS) and Sanitation in Developing Countries, Editors: Andreas Ulrich, Stefan Reuter and Bernd Gutterer, Authors: Bernd Gutterer, Ludwig Sasse, Thilo Panzerbieter and Thorsten Reckerzugel, Published by Water, Engineering and Development Centre, Loughborough University, UK.

D. IS : 5611 – 1987 (Reaffirmed 2002) – Code of Practice for Construction of Waste Stabilization Ponds (Facultative Type).

Input Data and Sewage Characteristics

1. Capacity of the STP = 2 MLD
2. Capacity of the STP = 2 x 1000 = 2000 m^3/day
3. Capacity of the STP = 2000 / (24 x 60 x 60) = 0.023148 m^3/s
4. Per capita water supply = 135 lpcd

As per Table 2.4 of CPHEEO's 'Manual on Water Supply and Treatment Systems (Drink from Tap)', March 2024, the per capital supply for cities having a population of less than 10 lakhs is 135 lpcd. Extract of this table is presented below as Table 2.12.

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The supply recommended supply should be at consumer end. This means that 15% system losses (NRW) shall be added to the demand.

Table 2. 11 Recommended maximum water supply levels in LPCD as per table 2.4 of CPHEEO's Manual on Water Supply and Treatment Systems, 2024

S. No.	Classification of towns/cities	Recommended Maximum water supply levels (lpcd)
1	Cities / towns with a population of less than 10 lakh (0.1 million)	135
2	Metro and Mega cities having a population of 10 lakh (1 million) or more	150

5. BOD_5 in the influent municipal sewage = 27 g / capital / day

As per Table 5.4 of CPHEEO Manual on Sewerage and Sewage Treatment Systems, the per capita contribution of BOD is 27 g /c / day.

6. Influent BOD_5 in the Anaerobic Pond = 250 mg/l

With 135 lpcd water supply and sewage generation at 80% of water supplied, the BOD concentration is obtained as:

BOD_5 = 27 x 1000 / (135 x 0.8) = 250 mg/l.

7. Influent BOD_5 in the facultative pond = 127.5 mg/l

The facultative pond is employed sequentially after the anaerobic pond. As per the design of anaerobic pond, the BOD removal efficiency is obtained as 49% at a temperature of 14.5^{o} C.

BOD removal efficiency (in %) = 2T + 20 = 2 x 14.5 + 20 = 49%. The reference for this has been dealt with in the anaerobic pond design section.

Balance BOD after anaerobic pond = 100 -49 = 51%

Influent BOD in the facultative pond = 0.51 x 250 = 127.5 mg/l.

8. Effluent BOD_5 in the treated water = 30 mg/l

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With reference to Table 5.3 of the CPHEEO manual, the permitted level of BOD_5 for discharge into inland surface water bodies is 30 mg/l. The table is based on 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.

9. Latitude of the City = 21.14^o N.

   The sample design is being done for a centrally located city in the country. The latitude for this is taken as 21.14 ^{o} N.

10. Lowest Average ambient temperature = 14.5^{o}C

    The lowest average ambient temperature corresponds to the coldest month (December) and the value is obtained from IMD. In the present study the temperature of 14.5^o C is assumed as representational value for latitude of 21.14^o N.

11. Altitude of the city = 310 m

    The data of average altitude of the city is taken from the ULB database / Survey of India database / one of the several websites listing Indian cities with their altitude. For the present study, the city lying at an altitude of 21.14^o N has an altitude of 310 m.

12. Permissible pond loading rate = 242.875 Kg/ Ha/ day

    The pond loading rate is obtained either from Table 1 of IS : 5611 – 1987 (Reaffirmed 2002) Indian Standard Code of Practice for Construction of Waste Stabilization Ponds (Facultative Type) or Table 5.14 of the CPHEEO manual, a copy of which is presented below. The table gives permissible organic loading rate at latitudes from 8^o N to 36^o N.

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Table 5.14 from CPHEEO Manual

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

In the present study the permissible pond loading rate for latitude of 21.14^{o} N is obtained through interpolation.

Permissible pond loading rate = 250 + (250-225) x (21.14 – 20) / (20 – 24) = 242.875 Kg / Ha/ day

13. Correction for Altitude

As per clause 5.8.4.4.1 of CPHEEO manual, the permissible pond loading rate is corrected for elevation of the city above mean sea level by dividing it by a factor given as:

1 + 0.003 x Elevation in meters = 1 + 0.003 x 310 = 1.93

Further, 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%. In the present study the sky is assumed to be clear for over 75% of the days in the year.

14. Corrected permissible pond loading rate = 242.875 / 1.93 = 125.8419 Kg / Ha / day.

15. Permissible pond loading rate based on temperature = 20T – 120

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As per Equation 5.36 of CPHEEO manual, the permissible pond loading rate based on temperature is given by:

$L_o = (20\ T - 120)$

Where,

Lo = design organic load in kg BOD5 / Ha /d and

T = average temperature during coldest month of the year in degree Celsius.

Lo = 2T – 120 = 20 x 14.5 – 120 = 170 Kg / Ha / day

16. Adopt a value of permissible pond loading rate which lies in between the permissible rates based on latitude + latitude or temperature. In the present study, the average of the two values is adopted.

    Adopted permissible pond loading rate = (125.8419 + 170) / 2.0

    = 147.921 Kg / Ha / day

17. The design permissible pond loading rate (say) = 148 Kg / Ha /day

18. BOD_5 loading rate of the town = average flow x Influent BOD / 1000

    = 2000 x 127.5 / 1000 = 255 Kg / ha /day

    Where 1000 is the conversion factor for gram to Kg.

19. Pond area required as per permissible loading rate = 255 / 148 = 1.7229 Ha

20. Depth of the pond = 1.5 m

    As per clause 5.8.4.4.3 of CPHEEO manual, '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.

    Hence the depth is taken as 1.5 m which is the maximum permissible. Lesser depth would increase the surface area for the same volume of the pond.

21. Pond volume = $1.7229 \times 10^4 \times 1.5 = 25844.5946\ m^3$

22. Pond detention time = 1.7229 x 10000 x 1.5 / 2000 = 12.9223 days, say 13 days

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23. Number of ponds in parallel = 2

As per clause 5.8.4.5.4 of CPHEEO manual, ‘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. 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.’

24. No. of ponds in series = 1

As per clause 5.8.4.5.4 of CPHEEO manual, one cell is provided in series after the two cells in parallel.

25. Total Number of ponds = 2 + 1 = 3 Nos.

26. BOD reaction rate constant $K_l = 0.2$

As per clause 5.8.4.4.2 of CPHEEO manual, ‘The value of K_l varies between 0.05 and 0.2 per day and is independent of temperatures above 15º C. The lower values were determined for secondary and tertiary ponds’.

27. Percentage reduction of BOD required = 76.47%

Reduction of BOD required = 127.5 – 30 = 97.5 mg/l

Percentage reduction required = 97.5 / 127.5 = 0.7647

28. Detention time required for plug flow conditions

Equation No. 5.37 of CPHEEO manual gives the detention time required for plug flow conditions in the pond

$\frac{L_e}{L_i} = e^{-K_l t}$ Equation 5.37 of CPHEEO manual

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Where.

Li and L e = influent and effluent BOD respectively

t = detention time

K_l = BOD reaction rate constant

(1-0.7647) = 0.2353 = $e^{-0.2(\frac{2t}{3}+\frac{t}{3})}$

t = ln (0.2353) / -0.2 = 7.2345 days

29. Detention time required for completely mixed flow conditions

Equation 5.38 of the CPHEEO manual gives the detention time required for completely mixed flow in the pond.

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

(1-0.7647) = $\frac{1}{[1+0.2\left(\frac{2t}{3}\right))(1+0.2\left(\frac{t}{3}\right)]}$

Solve to get t = 10.9353 days.

The detention time obtained in (22), i.e. t = 12.9223 days is more than 7.2345 days and 10.9353 days. Hence OK.

30. Population equivalent = 2 x 10^6 / 135 = 14815

31. Area of the facultative pond per person = 1.723 x 10^4 / 14815 = 1.163 m^2

32. Rate of sludge accumulation = 0.07 m^3 / person / year

As per clause 5.8.4.4.4 of CPHEEO Manual, '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.'

33. Sludge volume in one year = population equivalent x rate of sludge accumulation

= 14815 x 0.07 = 1037.03704 m^3

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34. Depth of first cell to accommodate the sludge is increased by 0.5 m.
Hence, depth of first cell = 1.5 + 0.5 = 2.0 m

35. No. of first cells allocated for sludge accumulation = 2

In case of parallel cells, there will be as many first cells allocated for sludge accumulation as there are cells in parallel.

36. Depth of sludge in the first 2 cells = 2.0 -1.5 = 0.5 m

The sludge depth can be varied based upon the desired interval of desludging. This will not affect the process dynamics.

37. Volume of sludge in the first 2 cells = (2/3) x (1.72297) x 10^4 x 0.5 = 5743.2333 m^3

Where 1.72297 is the total area, 0.5 m is the depth of sludge and 2/3 is for 2 parallel ponds out of three which are designed for holding sludge.

38. Duration of sludge cleaning = Volume allocated for sludge / volume of sludge per year

= 5743.2333/ 1037.0370 = 5.538 years. OK

39. Area of each pond = 1.72297 / 3 = 0.57432 ha

40. L/B ratio = 4

As per Table 5.13 of CPHEEO manual, the length to width ratio for D/UL value of 0.2 – 0.6 is 8:1or more. Since two cells are placed in series, the ratio for each cell is taken as 4:1, and the overall ratio becomes 8:1

41. Length L = $= \sqrt{4x0.57432x10000}$ = 151.5684 m, say 152 m

42. Breadth B = 177.6488 / 4 = 37.89209 m, say 38 m

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43. Frist 2 tanks size : 152 m x 38 m x 2.0 m

44. Subsequent tank size : 152 m x 38 m x 1.5 m

45. Length at mid-depth of primary and secondary tanks = 152 m

46. Width at mid-depth of primary and secondary tanks = 38 m

47. Side slope of pond (1V : 2 H) = 2

48. Top of embankment above mid-depth in primary tank = 2.0 m

49. Tank bottom below mid-depth in primary tank = 1.0 m

50. Top of embankment above mid depth in secondary tank = 1.75 m

51. Tank bottom below mid-depth in secondary tank = 0.75 m

52. Top length in primary tank = 152 + 2 x (2 x 2) = 160 m

53. Top width in primary tank = 38 + 2 x (2 x 2) = 46 m

54. Bottom length in primary tank = 152 – 2 x (2 x 1) = 148 m

55. Bottom width in primary tank = 38 - 2 x (2 x 1) = 34 m

56. Top length in secondary tank = 152 + 2 x (2 x 1.75) = 159 m

57. Top width in secondary tank = 38 + 2 x (2 x 1.75) = 45 m

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58. Bottom length in secondary tank = 152 – 2 x (2 x 0.75) = 149 m

59. Bottom width in secondary tank = 38 - 2 x (2 x 0.75) = 35 m

The dimensions of the primary and secondary facultative ponds in term of top, bottom and middle length and width are given in Table 2.13. The depth of all primary ponds is 2 m and that of secondary ponds is 1.5 m.

Table 2. 12 Summary of 1, 2, and 5 MLD Waste Stabilization Ponds

Capacity MLD	Component	No. of ponds	Length (m)	Width (B)	Depth (D)
1	Facultative Pond – Primary (Top / Middle / Bottom)	2	116/108/104	35/27/23	2
	Facultative Pond – Secondary (Top / Middle / Bottom)	1	115/108/105	34/27/24	1.5
2	Facultative Pond – Primary (Top / Middle / Bottom)	2	160/152/148	46/38/34	2
	Facultative Pond – Secondary (Top / Middle / Bottom)	1	159/152/149	45/38/35	1.5
5	Facultative Pond – Primary (Top / Middle / Bottom)	2	248/240/236	68/60/56	2
	Facultative Pond – Secondary (Top / Middle / Bottom)	1	247/240/237	67/60/57	1.5

2.5 Design of Anaerobic Baffled Reactor + Constructed Wetlands

In Class II to Class VI category towns, financial sustainability and availability of trained manpower is a major concern for O & M of STPs. In this scenario, there is a significant need to adopt reliable technologies which are simple to design, build and operate and at the same

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time are not OPEX intensive. The Anaerobic Baffled Reactor (ABR) is one such anaerobic process which has attracted many researchers due to design simplicity, use of non-sophisticated equipment, low excess sludge production, high treatment efficiency and low capital and operating costs.

Although the anaerobic process is efficient in the removal of organic material and suspended solids from municipal sewage, the effluent needs post-treatment for removing residual COD, nutrients, and pathogens. The post-treatment usually consists of an aerobic reactor which produces treated water meeting the effluent standards. There are a variety of wastewater purification methods that may be applied to fulfil the post treatment requirements. Constructed wet lands is one such technology which has been adopted in the present study for post-treatment of the ABR effluent.

2.5.1 Working of ABR

The Anaerobic Baffled Reactor (ABR) is a type of high-rate reactor which was developed at Stanford University as a form of series of up-flow anaerobic sludge blanket reservoirs by dividing the reactor into several compartments. A typical ABR consists of a settler followed by series of vertical baffles (4 to 8) that direct the wastewater under and over the baffles as it passes from the inlet to the outlet. This type of movement of swage through the reactor helps in retaining active biological mass even in the absence of fixed or suspended media. The bacteria within the reactor moves down the reactor horizontally at a low rate, giving rise to a SRT of 100 days at a HRT of 20 hours. Figure 2.5 shows the schematic representation of the ABR with a settler compartment.

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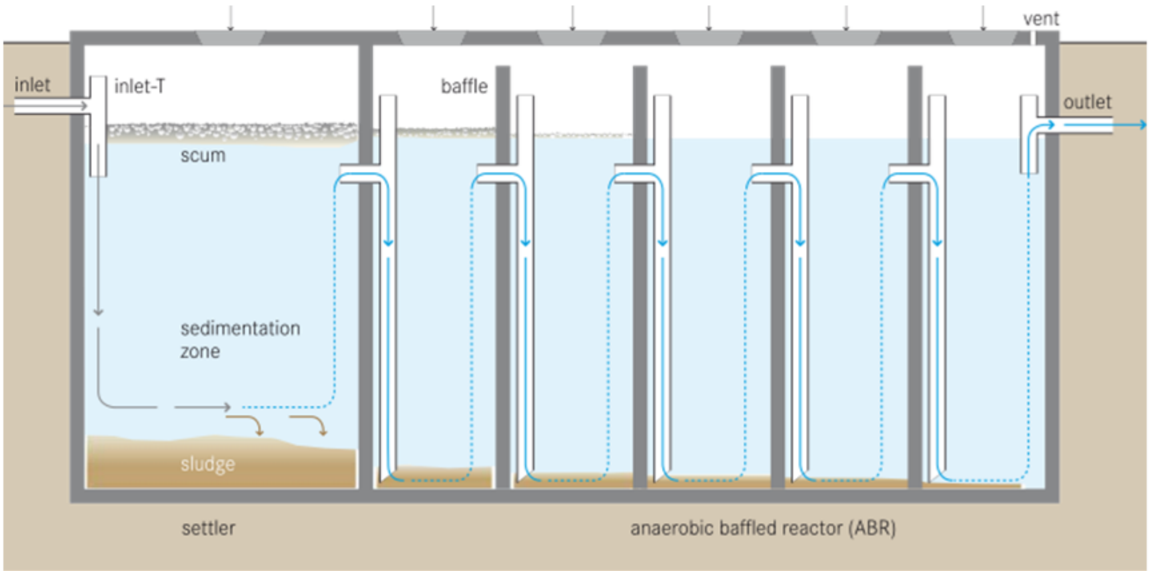

Figure 2. 4 Representation of the anaerobic settler and Anaerobic Baffled Reactor

(Source: https://commons.wikimedia.org/wiki/File:Anaerobic_Baffled_Reactor_ABR_diagram.svg)

2.5.2 Working of Constructed Wetlands

Use of constructed wetlands is a viable alternative treatment option which is suitable for small and medium sized communities in areas where land availability is not a constraint. Based on the subsurface flow, the constructed wetlands can be designed as either vertical flow or horizontal flow. In the present study, the horizontal flow constructed wetlands is adopted.

A constructed wetland is a shallow basin filled with some sort of filter material (substrate), usually sand or gravel, and planted with vegetation tolerant of saturated conditions. Wastewater is introduced into the basin and flows over the surface or through the substrate and is discharged out of the basin through a structure which controls the depth of the wastewater in the wetland.

During the water flow through the filter media, the wastewater will come in contact with a network of aerobic, anoxic and anaerobic zones. Attached and suspended microbial growth is responsible for the removal of soluble organic compounds through both aerobic and anaerobic biological action. The required oxygen for the process is obtained directly form the atmosphere by diffusion or oxygen leakage from the vegetation roots. Figure 2.6 shows the schematic representation of the constructed wetlands.

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The hydraulic loading can be up to 100 l / m^2 (Can be higher if course filter media is used) and the organic loading can be up to 10 g BOD / (m^2 day) because oxygen supply from the surface is limited. A slope of 1% is maintained along the flow direction. The height of the filter is usually 50-60 cm.

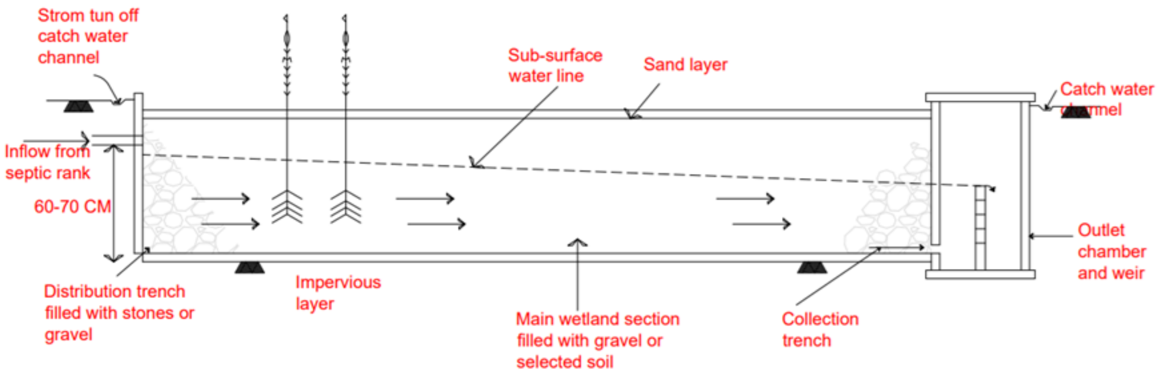

Figure 2. 5 Representation of a cross section through the Constructed Wetlands

2.5.3 Proposed Scheme of Treatment with ABR + Constructed Wetlands

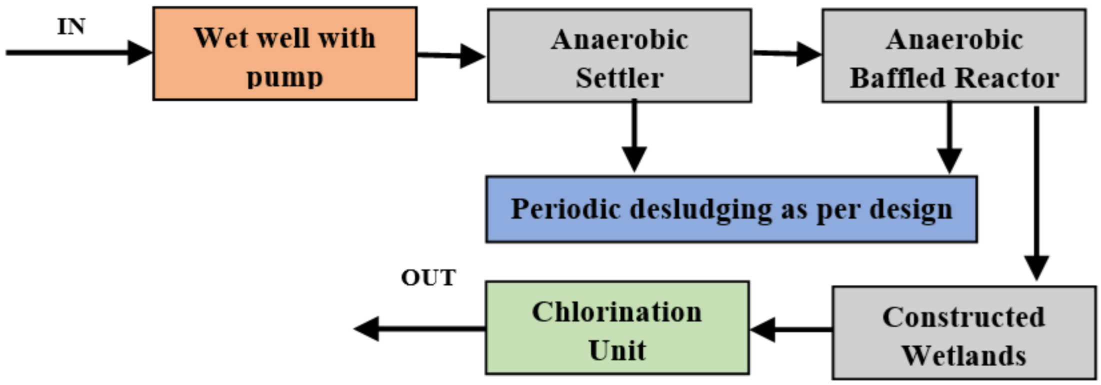

Figure 2. 6 Flow chart showing the unit processes in the ABR – Constructed Wetlands combination

The required effluent quality is proposed to be achieved in two stages, which are as follows:

i. Stage I: consists of Anaerobic settler followed by an Anaerobic Baffled Reactor

ii. Stage II: consists of constructed wetlands.

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The flow chart of the proposed scheme is shown in Figure 2.7. Since the area required for the constructed wetlands is large, it is desirable to adopt a modular concept for convenience of execution; with each module consisting of all the three units i.e., Anaerobic settler, Anaerobic Baffled Reactor and Constructed Wetlands. The unit is taken as 0.25 MLD. Hence, a combination of 4 units would produce a 1 MLD plant, 8 units would produce a 2 MLD plant and 20 units would be required for a 5 MLD plant. The design calculations of anaerobic settler and ABR is presented in Table 2.14 and the summary of 1 MLD, 2 MLD and 5 MLD ABRs is presented in Table 2.15.

Table 2. 13 Design calculations of 0.25 MLD Anaerobic Baffled Reactor

DESIGN OF 0.25 MLD ANAEROBIC SETTLER CUM ANAEROBIC BAFFLED REACTOR				
S. No.	**Design parameter**	**Notation / Formula used**	**Assumed / computed value**	**Units**
1	Daily wastewater flow	$Q_{average}$	0.25	MLD
2	Average flow m^3/day	$Q_{average}$ (MLD) x $10^6/10^3$	250	m^3/day
3	Peak factor	As per CPHEEO manual	3	unit less
4	Peak flow (m^3/day)	Q_{peak}	750	m^3/day
5	Peak flow (m^3/hour)	Q_{peak} (m^3/day)/24	31.25	m^3/hour
6	COD inflow	Given or assumed for municipal sewage	425	mg/l
7	BOD_5 inflow	Given or assumed for municipal sewage	250	mg/l
8	COD/BOD ratio	COD / BOD	1.7	ratio

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9	Settleable SS / COD ratio	chosen from the standard range of 0.35 to 0.45	0.42	ratio
10	Lowest digester temperature	For the chosen latitude of 22.14° N	14.5	°C
11	Desludging interval	Assumed based on recommended standard value	24	months
12	HRT in settler	Assumed based on recommended standard value	2	hours
13	COD removal rate in settler	Based on the COD removal in settlers graph	0.245	
14	COD / BOD removal factor	Based on the COD removal relative to wastewater strength in anaerobic filters graph	1.06	unit less
15	BOD_5 removal rate in settler	COD removal rate in settler *COD/BOD removal factor	0.26	
16	COD inflow into baffled reactor	COD inflow * (1-COD removal rate in settler)	320.88	mg/l
17	BOD_5 inflow into baffled reactor	BOD inflow *(1-BOD removal rate in settler)	185.08	mg/l
18	COD/BOD ratio after settler	COD inflow into baffled reactor/BOD inflow into baffled reactor	1.73	ratio
19	f-overload factor to calculate COD removal rate of anaerobic filter	Based on the COD removal affected by organic overloading graph	1	unit less
20	f-strength factor to calculate COD removal rate of anaerobic filter	Based on the BOD removal in relation to wastewater strength graph	0.90	unit less
21	f-temp factor to calculate COD removal rate of anaerobic filter	Based on COD removal in relation to temperature graph	0.56	unit less

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22	f-HRT percentage to calculate COD removal rate of anaerobic filter	Based on the graph BOD removal relative to HRT in baffled reactors	0.94	unit less
23	f-number of chambers in the ABR	Accounts for improvement in treatment as chambers increase up to 8	1.08	unit less
24	Theoretical removal rate according to above four factors	overloading x wastewater strength x temperature x HRT) factors	0.51	%
25	Practical COD removal rate	Computed by restricting unrealistic COD removal rate to 95%	0.51	%
26	COD out	(1-COD removal rate baffle only)*COD IN	157.61	mg/l
27	COD/BOD removal factor	Based on the COD removal relative to wastewater strength in anaerobic filters graph	1.09	unit less
28	Total COD removal rate	1-COD Out /COD inflow	0.63	%
29	Total BOD_5 removal rate	Total COD removal rate x COD/BOD removal factor	0.69	%
30	$BOD_{5\ OUT}$	(1-Total BOD removal rate) x BOD inflow	77.99	mg/l
31	Inner masonry dimensions chosen for settler width	Chosen keeping the width of the reactor in view	5	m
32	Inner masonry dimensions chosen for settler depth	Chosen based on recommended value	1.5	m
33	Sludge accumulation rate	Estimated based on the reduction of sludge volume during storage graph	0.0033	liter / g COD
34	Sludge volume	Total volume of sludge accumulated in the settler during the desludging period	38.80	m^3

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35	Volume of water	HRT x Peak flow	62.5	m^3
36	Total Volume of the settler	Sludge volume + water volume	101.30	m^3
37	Length of settler	Volume of settler / (Width x Depth)	13.51	m
38	Adopted length of the settler	Rounded off to nearest upper quarter m	13.5	m
39	Provided surface area of the settler	Adopted length x adopted width	67.5	m^2
40	Surface load	Ratio of Peak flow to the surface area	0.46	m^3/m^2
41	Required inner length of first chamber of settler	Length corresponding to 2/3rd of the settler volume	9	m
42	Required inner length of second chamber of settler	Length corresponding to 1/3 of the settler volume	4.5	m
43	Maximum upflow velocity in the ABR	Adopted in the standard range of 1.4-2.0 m/hour	1.8	m/hour
44	Number of upflow chambers	Chosen in the range of 4-8 such that HRT is satisfactory	6	No.
45	Depth at outlet	Chosen based on recommended value	1.5	m
46	Length of chamber (Calculated)	Should not exceed half depth (Depth at outlet / 2)	0.75	m
47	Length of chamber (Adopted)	Rounded off to nearest upper quarter m	0.75	m
48	Area of single upflow chamber	Peak flow/upflow velocity	17.36	m^3
49	Width of chamber (calculated)	area of single chamber / Length of the chamber	23.15	m

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50	Width of chamber (Adopted)	Rounded off to nearest upper quarter m	23.5	m
51	Actual upflow velocity	Peak hourly flow/(chamber length x chamber width)	1.77	m/hour
52	Width of down flow shaft	Chosen based on usually adopted value	0.25	m
53	Actual volume of baffled reactor	(width of down flow shaft + length of chamber) x No. of upflow chambers x depth at outlet x width of chamber	211.5	m^3
54	Actual total HRT	Reactor volume x 24 / Daily average flow	19.34	hours
55	Organic loading of BOD_5	BOD inflow into reactor x hourly peak flow x 24 / (volume of reactor x 1000)	0.66	$kg/m^3/day$
56	Organic loading of COD	BOD inflow into reactor x hourly peak flow x 24 / (volume of reactor x 1000)	1.14	$kg/m^3/day$
57	Biogas production	(COD inflow - COD removal rate) x Q average x 0.35/1000/0.7/0.5	16.71	m^3/day

Green cells: Input variable data to be provided Yellow cells: standard data from CPHEEO manual or other standards Red Cells: Design output

2.5.3.1 Design Steps of Anaerobic Baffled Reactor

REFERENCES FOLLOWED:

A. Decentralized Wastewater Treatment Systems (DEWATS) and Sanitation in Developing Countries, Editors: Andreas Ulrich, Stefan Reuter and Bernd Gutterer, Authors: Bernd Gutterer, Ludwig Sasse, Thilo Panzerbieter and Thorsten Reckerzugel, 2009, Published by Water, Engineering and Development Centre, Loughborough University, UK.

B. Online workbook titled 'Decentralized Liquid Waste Management - Design Workbook', National Institute of Urban Affairs (NIUM) and College of Engineering Pune, 2019.

C. 'Waste Stabilization Ponds and Constructed Wetlands Design Manual', S. Kayombo, T.S.A. Mbwette, J.H.Y. Katima, University of Dar es Salaam and N. Ladegaard, S.E. Jorgensen, Danish University of Pharmaceutical Sciences Section of Environmental Chemistry, Copenhagen, Denmark.

D. Wastewater Treatment for Pollution Control and Reuse, third edition, Soli J Arceivala and Shyam R Asolekar, Mc Graw Hill Education, 2007.

E. Compendium of Sanitation Systems and Technologies, 2nd revised edition, Elizebeth Tilly, Lukas Ulrich, Christoph Luthi, Philippe Remond and Christian Zurbrugg, 2014, Eawag : Swiss Federal Institute of Aquatic Science and Technology, Switzerland and The International Water Association (IWA).

F. DEWATS – Decentralized Wastewater Treatment in Developing Countries, Ludwig Sasse, 1998, BORDA, Bremen Overseas Research and Development Association.

G. Anaerobic Baffled Reactor (ABR) Design Considerations for Faecal Sludge, 2021, UPM Umwelt-Projekt-Management GmbH.

Input Data and Sewage Characteristics

1. Design flow of sewage (average flow) = $Q_{average}$ = 0.25 MLD

2. Average flow = $Q_{average} \times 10^6 / 10^3 = 0.25 \times 10^6 / 10^3 = 250\ m^3/day$

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3. Peak factor = 3.0

The present design is undertaken for a city with a population < 20, 000 as per census 2011. As per clause 3.5 of CPHEEO manual the peak factor for this population is 3.00. The peak factor for different populations is given in Table 3.2 of the CPHEEO Manual.

Table 3.2 from CPHEEO Manual

Table 3.2 Peak factor for Contributory Population

Contributory Population	Peak Factor
up to 20,000	3.00
Above 20,001 to 50,000	2.50
Above 50,001 to 7,50,000	2.25
above 7,50,001	2.00

Source: CPHEEO, 1993

4. Peak flow = Q_{peak} = 3.00 x 250 = 750 m^3/day

5. Peak flow = Q_{peak} = 750 / 24 = 31.25 m^3/hour

6. COD of influent = 425 mg/l

The value of COD of influent sewage is taken from Table 5.4 of CPHEEO manual

7. BOD_5 inflow = 250 mg/l

The value of BOD_5 of influent sewage is taken from Table 5.4 of CPHEEO manual

8. COD / BOD ratio = 425 / 250 = 1.7

9. Settleable SS / COD ratio = 0.42

As per the Rules of thumb for design from Reference B, the SS/COD ratio for Domestic sewage is in the range of 0.35-0.55, usually 0.42

10. Lowest digester temperature = 14.5^o C

The lowest temperature of the year corresponds to the month of December for the chosen latitude of 22.14^o. The value of 14.5^o C is obtained from IMD data. Similar data can be obtained for other cities under consideration.

11. Desludging interval = 24 months

As per S.10 of reference E, 'maintenance is limited to the removal of accumulated sludge and scum every 1 to 3 years. Hence 24 months is OK.

Similarly, as per the Rules of Thumb of reference B, the collection of sludge is to be done every 18 – 36 months.

12. Hydraulic Retention Time (HRT) in the settler = 2 hours.

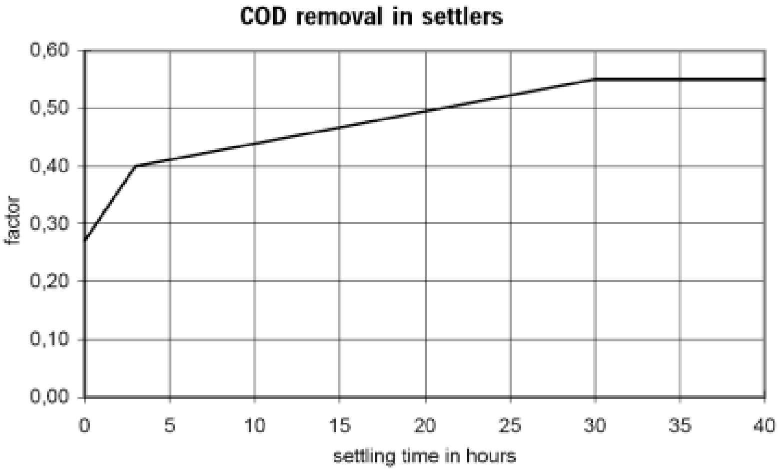
As per clause 8.8 of CPHEEO manual, 'Pre-treatment settler: retention time of about 2 hours, BOD reduction of about 30%.'

Also as per the Rules of Thumb of reference B, the HRT in settler for DEWATS is 2 h.

13. COD removal rate in settler is obtained from the graph in Reference F for HRT < 3.

COD removal rate factor = (HRT – 1) x 0.1/2 + 0.3

= (2 – 1) x 0.1 / 2 + 0.3 = 0.35



HRT < 1:
factor = HRT*0.3

HRT < 3:
factor = [(HRT-1) *0.1/2] +0.3

HRT < 30:
factor = [(HRT-3) *0.15/27] +0.4

HRT ≥ 30:
factor = 0.58

Figure 68 of Reference F

Based on the design parameters in Reference B, surface load must be less than 0.6 m^3/m^2.

COD removal rate (%) = $\frac{\frac{SS}{COD_{ratio}}}{Surface\ load} x factor\ HRT$

$$= \frac{0.42}{0.6} x 0.35 = 0.245, i.e. 24.5\%$$

14. COD / BOD removal rate in settler is obtained from the graph in Reference F, reproduced below for ready reference.

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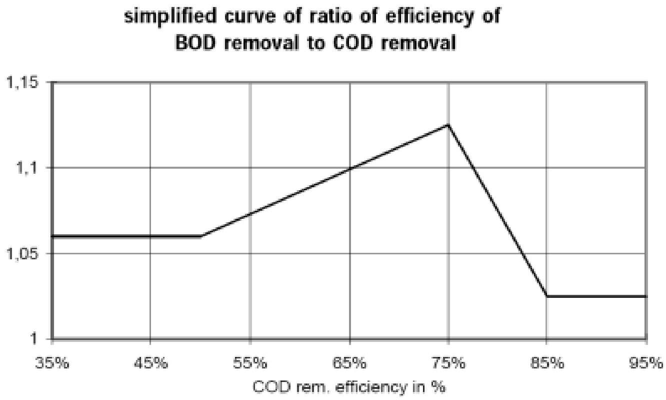


COD_{rem} < 0.5:
factor = 1.06

COD_{rem} < 0.75:
factor = [(COD_{rem}-0.5) *0.065/0.25] +1.06

COD_{rem} < 0.85:
factor = 1.125-[(COD_{rem}-0.75) *0.1/0.1]

$COD_{rem} \geq$ 0.85:
factor = 1.025

Figure 65 of Reference F

For COD removal rate of 0.245 < 0.5, COD / BOD removal factor = 1.06.

15. BOD_5 removal rate in settler = COD removal rate in settler x COD / BOD removal factor = 0.245 x 1.06 = 0.2597, i.e. 25.97%.

16. COD inflow in to the baffled reactor = COD inflow x (1 – COD removal rate in settler) = 425 x (1 – 0.245) = 320.875 mg/l.

17. BOD_5 inflow into the baffled reactor = BOD_5 inflow x (1 – BOD removal rate in settler) = 250 x (1 – 0.2597) = 185.075 mg/l

18. COD / BOD ratio after anaerobic settler = COD inflow into baffled reactor / BOD inflow into baffled reactor = 320.875 / 185.075 = 1.73376

19. Determine factor for organic overloading in the tank from the graph in reference F.

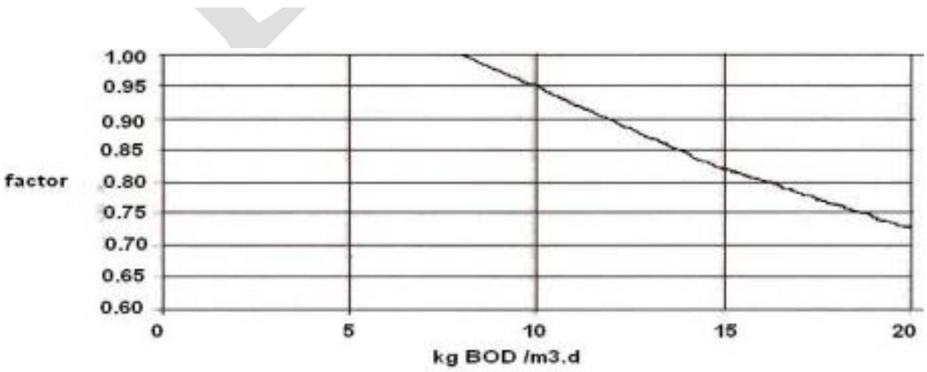


Figure 77 of Reference F

Organic Load < 8

Factor = 1.0

Organic Load < 15
Factor = 1 - (organic load – 8) x 0.18/7

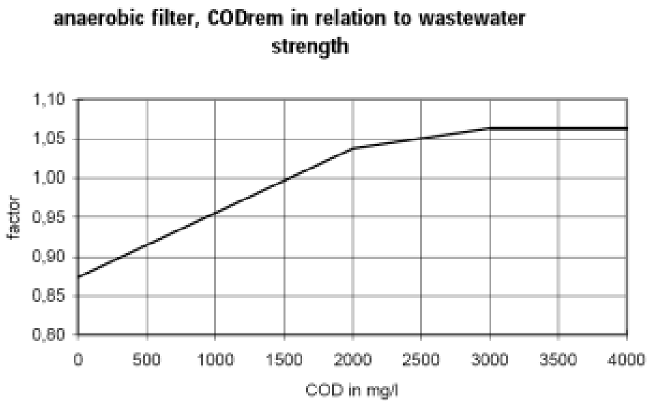
Organic load > 15
Factor = 0.82 – (organic load – 15) x 0.9 /5

Organic loading of BOD_5 = 1.3785641 $Kg/m^3/day$ obtained from Row No. 56 of Table 6.12.

Since Organic loading is < 8 Kg / m^3 / day, factor = 1.0

20. Determine factor for strength of sewage.

The factor for wastewater strength is computed using the graph given in Reference B.



COD_{in} < 2000 mg/L:
factor =([COD_{in}*0.17]/2000) +0.87
COD_{in} < 3000 mg/L:
factor = ([COD_{in} – (2000*0.02)]/1000) +1.04
COD_{in} ≥ 3000 mg/L:
factor = 1.06

Figure 10 of Reference B

COD_{IN} in the ABR = 320.875 < 2000.

Hence, factor = 320.875 x 0.17 / 2000 +0.87 = 0.89727

21. Determine factor for temperature

The factor for COD removal relative to temperature is obtained from the graph presented in Reference F.

Temperature = 14.5^o C which is < 20^o C

Temperature factor = (14.5-10) x 0.39 / 20 + 0.47 =0.55775

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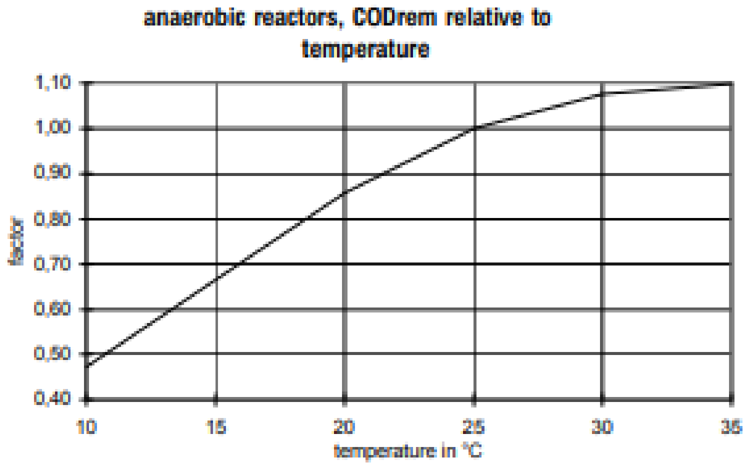


Temp < 20° C:
Factor= (temp-10) *0.39 /20+ 0.47

Temp < 25° C:
Factor= (temp-20) *0.14/5+ 0.86

Temp < 30° C:
Factor= (temp-25) *0.08/5+1.0

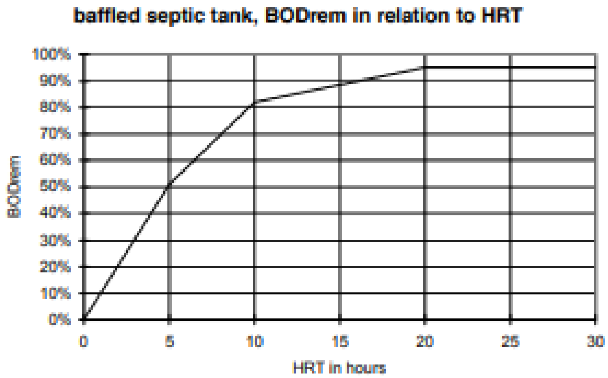
Temp < 35° C:
Factor= (temp-30) *0.06/5+1.08

Temp ≥ 35° C:
Factor= 1.1

Figure 72 of Reference F

22. Determine factor for HRT

The factor for HRT is obtained from the graph presented as Figure 76 in Reference F.



HRT < 5h
Factor = HRT*0.5/5

HRT < 10h
Factor = [(HRT-5)*0.32/5] + 0.5

HRT < 20h
Factor =[(HRT-10)*0.13/10] + 0.82

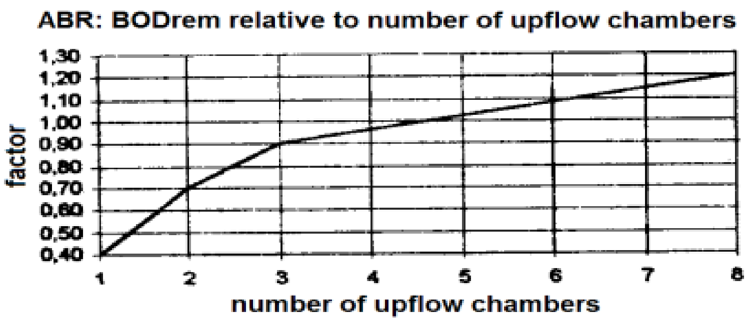
HRT > 20h Factor = 0.95

Figure 76 of Reference F

Actual HRT = 19.3371 hours < 20 hours

HRT factor = (19.3371 – 10) x 0.13 / 10 + 0.82 = 0.94138

23. Determine factor for number of chambers by using the graph in Reference B.



no = 1:
factor =0.4
no = 2:
factor =0.7
no = 3:
factor =0.9
no > 3:
factor =(no-3) *0.06+0.9

Figure 14 of Reference B

In the present study, No. of chambers = 6

Factor for number of chambers = (6 – 3) x 0.06 + 0.9 = 1.08

24. Theoretical COD removal rate expressed as percentage = product of the factors for overload, strength, temperature and HRT

= 1 x 0.89727 x 0.55775 x 0.94138 x 1.08 = 0.50881%

25. Determine the practical COD removal rate

The theoretical COD removal rate is obtained by using performance curves which are based on tests in laboratory and subsequently extrapolated. The maximum value of strength, temperature and chambers factors is more than 1. This can result, sometimes, in a performance value higher than 100%. In this stage of design, the treatment efficiency is restricted by specifying a standard limit to arrest unrealistic BOD removal rates.

Limit specified in Reference G is 95%.

Limit specified in References A, B is 98%

In the present study, the limit of 95% is adopted. This can be easily changed in the spread sheet to 98% or any other value without much effort.

Theoretical COD removal rate = 0.50881% < 0.95

Hence practical COD removal rate = 0.50881%

26. COD_{out} = (1-COD removal rate)*COD_{IN}

= (1 - 0.50881) x 320.875 = 157.61059 mg/l

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27. COD / BOD removal rate in reactor is obtained from the graph in Reference F, reproduced below for ready reference.

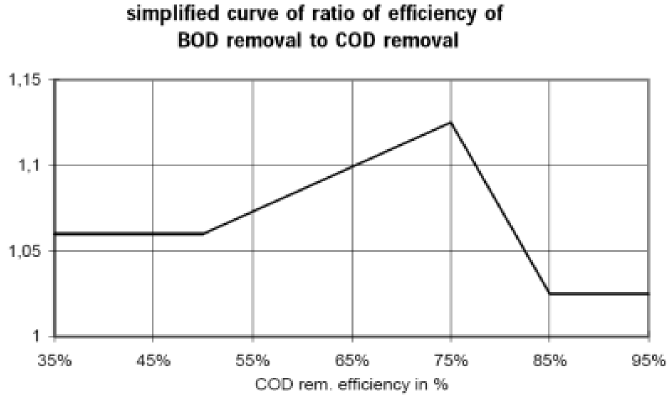

Figure 65 of Reference F

$COD_{rem} < 0.5$:
factor = 1.06

$COD_{rem} < 0.75$:
factor = [(COD_{rem}-0.5) *0.065/0.25] +1.06

$COD_{rem} < 0.85$:
factor = 1.125-[(COD_{rem}-0.75) *0.1/0.1]

$COD_{rem} \geq 0.85$:
factor = 1.025

For total COD removal rate of 0.62915 < 0.75, COD / BOD removal factor = (0.62915 – 0.5) x 0.065 / 0.25 + 1.06 = 1.09358

28. Total COD removal rate = 1 – COD_{Out} / COD_{In} = 1 – 157.61059/425 = 0.62915

29. Total BOD_5 removal rate = Total COD removal rate x COD/BOD removal factor
= 0.62915 x 1.09358 = 0.68802

30. $BOD_{5\ Out}$ = (1 – total BOD removal rate) x BOD_{In} = (1 – 0.68802) x 250 = 77.995 mg/l

31. Select Settler width = 5.0 m
This is the inner masonry dimension for settler width.
The settler width is chosen based upon the site conditions.

32. Select settler depth = 1.5 m
This is the inner masonry dimension for the settler depth.
In most of the designs (References A, F, G) the depth of the settler is taken as 1.5 m. There is no explicit mention on the standard depth to be adopted.

33 Sludge accumulation rate

The reduction of sludge volume with time is obtained using the graph in Reference B.

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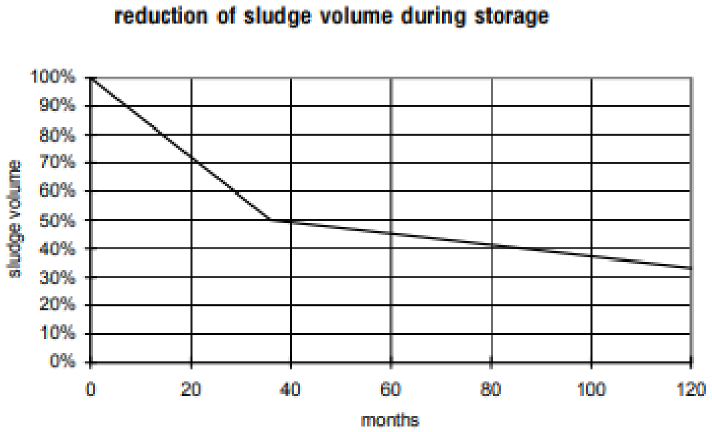

SRT< 36:
factor = 1-(SRT*0.014)

SRT< 120:
factor = 0.5-(SRT-36) *0.002

SRT≥ 120:
factor = 1/3

Figure 67 of Reference F

SRT = Desludging interval = 24 months

The sludge volume in litres per g BOD removal is 0.005 litre sludge / g BOD as per Rules of thumb in Reference B

Sludge accumulation rate = 0.005 x (1 – 24 x 0.014) = 0.00332 litre / g BOD

34. Determine the sludge volume

Volume of sludge = sludge accumulation rate (litre/g) x {(BOD_{IN} – BOD_{OUT})}/1000 in g/l x SRT (months) x 30 (days) x average flow (m^3/day)

= 0.00332 x (250 – 185.075) x 24 x 30 x 250 / 1000 = 38.79918 m^3

35. Water volume

Determine the water volume by considering the peak flow.

Water volume = HRT (hours) x peak flow (m^3/h) = 2 x 31.25 = 62.5 m^3

36. Settler volume = sludge volume + water volume = 38.79918 + 62.5 = 101.29918 m^3

37. Length of the settler = Volume/(width x depth) = 101.29918/(5 x 1.5) = 13.50656 m

As per the Rules of thumb in Reference B, the recommended length to width ratio ranges from 3:1 to 2:1.

The choice of width (5 m) and length (13.5 m) provides a length to width ratio of 2.7:1 which fall in the recommended rang. While designing, after a first trail the required dimensions can be obtained to comply with the desired ratio.

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38. Adopted length of the settler = 13.5 m

39. Provided surface area of the settler = width x length = 5 x 13.5 = 67.5 m^2

40. Surface load = peak flow / surface area = 31.25 / 67.5 = 0.46296 m^3 / m^2
As per the Rules of thumb for design of anaerobic settler in Reference B, the recommended surface load is 0.6 m^3 / m^2 of wastewater peak flow
Hence the surface load of 0.46296 < 0.6 is OK.

41. Required inner length of the first chamber of the settler = 2/3 of the total length = 2 x 13.5 / 3 = 9 m
As per the Rules of thumb in Reference B, the first chamber is 2/3 of the total length

42. Required inner length of the second chamber of the settler = 1/3rd of the total length = 1 x 13.5 / 3 = 4.5 m.

43. Maximum upflow velocity = 1.8 m/hour
As per the Design spread sheet of reference A, the recommended range of upflow velocity is 1.4 to 2 m/hour.
As per reference F also the recommended range of upflow velocity is 1.4 to 2 m/hour.
As per the Rules of thumb in Reference B, the up-flow velocity is 0.9 to 1.2 m/hour.
In the present design a velocity of 1.8 m/hour has been adopted. Values lesser than this can be adopted, if warranted based on evidence of improved quality, but the system will become bigger and hence expensive.

44. Number of upflow chambers = 6
As per all available references, the number of chambers can be varied between 4 and 8. The minimum number is 4 to obtain a reasonable degree of treatment and beyond 8 chambers there is no evidence of improvement in the treatment level.
As per reference G, the minimum chambers is 3 and maximum is 6. If calculations result in more than six chambers, two or more ABRs are recommended in parallel.
In the present design 5 or 6 upflow chambers have been found to be adequate.

45. Depth at outlet = 1.5 m.

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The outlet depth is kept the same as in the chambers. The outfall pipe of the ABR should then be directed to the next stage of treatment.

46. Length of the chamber of the ABR = outlet depth / 2 = 1.5 / 2 = 0.75 m.
As per Rules of thumb in reference B, the ideal lengths of chambers are taken as 0.70 to 0.85 m.

47. Adopted length of chamber = 0.75 m which is same as the computed length.

48. Area of single upflow chamber = peak flow/upflow velocity = 31.25/1.8 = 17.3611 m^2.

49. Computed width of the chamber = Area of a single chamber / length of the chamber
= 17.3611 / 0.75 = 23.14815 m.

50. Adopted width of chamber = 23.5 m.

51. Actual upflow velocity = Peak flow/chamber area = 31.25/(23.5 x 0.75) = 1.77305 m/h.

52. Width of the down flow shaft 0.25 m. (Assumed the usually adopted value in various designs)

53. Actual volume of baffled reactor = Volume computed by considering the down flow shaft = (0.25 + 0.75) x 6 x 1.5 x 23.5 = 211.5 m^2.

54. Actual total HRT = Reactor Volume x 24 / daily average flow
= 211.5 x 24 / (250 x 1.05) = 19.33714 hours

As per rules of thumb of reference B, the HRT should not be less than 8 hours, better between 12 -14 hours for the whole system. Above 20 hours, reduction is very minimal and not any more economically viable. 20 h ours would be the optimum HRT.

The computed HRT is reduced by 5% in order to account for the presence of sludge.

55. Organic loading for BOD = BOD_{IN} into the ABR x peak flow x 24 / Volume of reactor
= 185.075 x 31.25 x 24 /(211.5 x 1000) = 0.6563 Kg / m^3 / day

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56. Organic loading for COD = COD_{IN} into the ABR x peak flow x 24 / Volume of reactor = 320.875 x 31.25 x 24 /(211.5 x 1000) = 1.13785 Kg / m^3 / day

As per Rules of thumb in reference B, the organic loading should be < 6 Kg/m^3/day BOD.

As per clause 9.2.5 of reference A, 'The organic load should be below 3.0 kg COD / m^3 /day.' As per reference G, 'BOD/COD load Max. 3 kg COD / d per m^3 of ABR; low loads give better performances.'

The obtained values of 0.6563 kg/ m^3 / day and 1.13785 Kg/m^3 /day comfortably meets the criteria stated for both BOD and COD.

57. Biogas production = (425 – 157.6108) x 250 x (0.35/1000) x 0.7 x 0.5 = 8.1888 m^3/day As per reference G, theoretically, 350 litre (0.35 m^3) of methane gas is produced per Kg of COD removed based on molar calculations. The efficiency of gas removal is usually 70% and 50% of the produced gas is in dissolved form.

As per reference C, 'Methane production virtually ceases below temperatures of 15^0 C. Thus, in areas where the pond temperature remains below 15^0 C for more than a couple of months of the year, careful consideration should be given to deciding whether or not anaerobic units are needed.' In the present design, the temperature remains at 14.5^o C only for the coldest period in December. Hence anaerobic process can be expected to work satisfactorily during the remaining 11 months.

2.5.3.2 Data Entry Sequence and Guidance for ABR spread sheet

The spread sheet for design of anaerobic settler cum baffled reactor has data cells marked in green colour which need to be filled up in order obtain the design. The sequence of data entry is as follows:

1. Enter data sequentially in cell Nos. 1, 3, 6, 7, 9, 10, 11, 12

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2. Choose appropriate values for settler width and depth in cells 31 and 32. The width depends on the site conditions and the planned number of units.

3. Next enter the adopted length in cell 38. Choose a value such that the surface load in cell 40 is < 0.6 m^3/m^2. The computed value obtained in cell 37 can be rounded off and entered in cell 38 provided the criterion of cell 40 is satisfied.
4. Enter the values in cells 43, 44 and 45. The number of chambers in cell 44 can be varied such that the HRT of cell 54 is less than 20 hours and organic loading of cell 55 is less than 3 $Kg/m^3/day$.

5. Choose suitable value for cell 47. The depth is mostly 1.5 m, in which case this cell will be 0.75 m.

6. Enter the adopted width of the chamber in cell 50. This may be rounded off value obtained in cell 49 or a new value, if so dictated, by site conditions. Changes in adopted width changes the HRT and the organic loading.

Table 2. 14 Summary of 1, 2 and 5 MLD Anaerabic Bafffled Reactor

Unit Capacity (MLD)	**Component**	**No.**	**L (m)**	**B (m)**	**D (m)**
0.25	Anaerobic Settler	1	13.5	5.0	1.5
	Anaerobic Baffled Reactor	1	6.0	23.5	1.5
S. No.	Component	No. of Units			Remarks
		1 MLD	2 MLD	5 MLD	
1	Anaerobic Settler	4	8	20	The 0.25 MLD module can be replicated as per site conditions
2	Anaerobic Baffled Reactor	4	8	20	

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The Anaerobic settler cum Baffled Reactor is followed up by an aerobic process like constructed wetland. Table 2.16 presents the design of 0.25 MLD constructed wetland. The design of higher unit, if required, can be carried out in a similar manner.

The proposed arrangement of anaerobic settler, anaerobic baffled reactor and constructed wetlands for 0.25 MLD flow is shown in Figure 2.8. The possible arrangement of the 0.25 MLD module to produce 1, 2, 3 or 4 MLD capacity is shown in Figure 2.9.

Table 2. 15 Design calculations of 0.25 MLD Constructed Wetlands

DESIGN OF 0.25 MLD HORIZONTAL GRAVEL FILTER CONSTRUCTED WETLAND				
S. No.	**Design parameter**	**Notation / Formula used**	**Assumed / computed value**	**Units**
1	Design flow of Sewage	$Q_{average}$	0.25	MLD
2	Average flow in m^3/day	$Q_{average}$ (MLD) x $10^6/10^3$	250	m^3/day
3	BOD of the influent waste water	Corresponds to the BOD of the effluent from the ABR	78	mg/l
4	BOD of the effluent from the constructed wet land	CPHEEO norms	30	mg/l
5	Temperature	Minimum temperature of the year	14.5	^oC
6	Height of the filter	Adopted standard value	0.6	m
7	Slope of the wetland, S	Adopted standard value	1	%
8	Voids in the gravel (Porosity)	Vide Table 3.4 of US EPA manual	39	%
9	Ks, Hydraulic conductivity of the medium (Coarse Sand)	Vide Table 3.4 of US EPA manual	480	m^3/m^2-day
10	Size of gravel / grains	Vide Table 3.4 of US EPA manual maximum 10% grain size	2	mm
11	K_{20} = Rate constant at 20^{o}C	Vide Table 3.4 of US EPA manual	1.35	DL
12	Check the product (K_s S)	Should be < 8.6	4.8	OK

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13	K_T = Rate constant at design temperature	$K_T = K_{20} (1.1)^{T-20}$	0.799234091	DL
14	Required cross sectional area of the bed	$A_c = Q / (K_s S)$	52.08333333	m^2
15	Determine the bed width, W	$W = A_c / d$	86.80555556	m
16	Adopted bed width, W		87.00	3
17	Required surface area	$A_s = (Q (\ln C_o - \ln C_e)) / (K_T d n)$	1277.279802	m^2
18	Determine the bed length L	$L = A_s / W$	14.71426332	m
19	Adopted bed length, L		15.00	m
20	Hydraulic Retention time	$V_v / Q = L W d n / Q$	1.195533894	days

Green cells: Input variable data to be provided Yellow cells: standard data from CPHEEO manual or other standards Red Cells: Design output

The summary of design of 0.25 MLD ABR + Constructed wetlands is presented in Table 2.17. The proposed arrangement of anaerobic settler, anaerobic baffled reactor and constructed wetlands for 0.25 MLD flow has been shown in Figure 2.8. The possible arrangement of the 0.25 MLD module to produce 1 or 4 MLD capacity is shown in Figure 2.9.

2.5.3.3 Design Steps of Constructed Wetland

REFERENCES FOLLOWED:

A. Design Manual – Constructed Wetlands and Aquatic Plant Systems for Municipal Wastewater Treatment, U.S. Environmental Protection Agency, September 1988.

B. Decentralized Wastewater Treatment Systems (DEWATS) and Sanitation in Developing Countries, Editors: Andreas Ulrich, Stefan Reuter and Bernd Gutterer, Authors: Bernd Gutterer, Ludwig Sasse, Thilo Panzerbieter and Thorsten Reckerzugel, 2009, Published by Water, Engineering and Development Centre, Loughborough University, UK.

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C. Online workbook titled 'Decentralized Liquid Waste Management - Design Workbook', National Institute of Urban Affairs (NIUM) and College of Engineering Pune, 2019.

D. 'Waste Stabilization Ponds and Constructed Wetlands Design Manual', S. Kayombo, T.S.A. Mbwette, J.H.Y. Katima, University of Dar es Salaam and N. Ladegaard, S.E. Jorgensen, Danish University of Pharmaceutical Sciences Section of Environmental Chemistry, Copenhagen, Denmark.

E. Wastewater Treatment for Pollution Control and Reuse, third edition, Soli J Arceivala and Shyam R Asolekar, Mc Graw Hill Education, 2007.

F. Compendium of Sanitation Systems and Technologies, 2nd revised edition, Elizebeth Tilly, Lukas Ulrich, Christoph Luthi, Philippe Remond and Christian Zurbrugg, 2014, Eawag : Swiss Federal Institute of Aquatic Science and Technology, Switzerland and The International Water Association (IWA).

G. DEWATS – Decentralized Wastewater Treatment in Developing Countries, Ludwig Sasse, 1998, BORDA, Bremen Overseas Research and Development Association.

H. Constructed Wetlands Manual, United Nations Human Settlements Programme (UN-HABITAT), 2008.

Input Data and Sewage Characteristics

1. Design flow of sewage (average flow) = $Q_{average}$ = 0.25 MLD

2. Average flow = $Q_{average} \times 10^6 / 10^3 = 0.25 \times 10^6 / 10^3 = 250\ m^3/day$

3. BOD of the influent wastewater

   The influent into the horizontal gravel filter (constructed wetlands) is the effluent of the ABR. From the ABR design, the BOD is effluent is obtained as 77.9934 mg/l, say 78 mg/l. This is the BOD in the influent to the horizontal gravel filter.

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4. BOD of the effluent from the constructed wetland = 30 mg/l

With reference to Table 5.3 of the CPHEEO manual, the permitted level of BOD_5 for discharge into inland surface water bodies is 30 mg/l. The table is based on 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.

5. Temperature = 14.5^o C

The lowest average ambient temperature corresponds to the coldest month (December) and the value is obtained from IMD. In the present study the temperature of 14.5^o C is assumed as representational value for latitude of 21.14^o N.

6. Height of filter, d = 0.6 m

As per section 3.3.2 of the US EPA Manual (Reference A), the height of the filter is determined by the type of proposed plantation. “The major oxygen source for the subsurface components (soil, gravel, rock, and other media, in trenches or beds) is the oxygen transmitted by the vegetation to the root zone. In most cases the subsurface flow system is designed to maintain flow below the surface of the bed, so there can be very little direct atmospheric reaeration. The selection of plant species is therefore an important factor.

Work at the pilot wetlands in Santee, California, indicated that most of the horizontally growing root mass of cattails was confined to the top 300 mm of the profile. The root zone of reeds extended to more than 600 mm and bulrushes to 760 mm. In the cooler climate of Western Europe the effective root zone depth for reeds is also considered to be 600 mm.”

As per Table 30 of Reference B, the depth of filter varies from 0.3 to 0.6 m.

As per the rules of thumb in reference C, the height of filter is 50 – 60 cm.

As per clause 9.10 of reference G, ‘For this reason, the filter bed should not be deeper than the depth to which plant roots can grow (30 - 60 cm) as water will tend to flow faster below the dense cushion of roots. However, treatment performance is generally

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best in the upper 15 cm due to oxygen diffusion from the surface. Therefore, shallow filters are more effective compared to deeper beds of the same volume

It may be noted that in most of the cited references, as per rule of thumb, the recommended height of the filter is between 30 and 60 cm. In the present design, the height of filter is taken as 0.6 m on the assumption that reeds will be planted. It may be understood that Reed is a common name for several tall, grass-like plants of wetlands. The height of the filter may be varied if warranted by the type of plant selected.

7. Slope of the wetland = 1%

As per clause 9.2.7.1 of Reference B, 'While the top of the filter is kept strictly horizontal to prevent erosion, the bottom slopes down from inlet to outlet, ideally at 1%. Site conditions permitting, bigger slope is also possible.'

As per the rules of thumb in reference C, the slope is 1%.
As per 9.10 of reference G, the same statement as in reference A is repeated.
As per section 3.3.2 of Reference A, 'The bed slope is based on the site topography. Most systems have been designed with slope of 1 percent or slightly higher.'

8. Voids in the gravel = 39%
As per clause 9.2.7.1 of reference B, 'Gravel has 30 to 45% voids, depending on size and shape. Size of gravel = 5-7 mm, 10-12 mm, 50-70 mm diameter of the gravel.
As per the rules of thumb in reference C, the voids of gravel fall in the range of 35% - 45%.

Table 3.4 of Reference A presents the media characteristics for subsurface flow systems. The table gives maximum grain size, porosity, Hydraulic

Table 3.4 of Reference A

Table 3-4. Media Characteristics for Subsurface Flow Systems

Media Type	Max. 10% Grain Size, mm	Porosity (n)	Hydraulic Conductivity (k_s), m^3/m^2-d	K_{20}
Medium Sand	1	0.42	420	1.84
Coarse Sand	2	0.39	480	1.35
Gravelly Sand	8	0.35	500	0.86

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conductivity and K_{20} values for three media types; namely, Medium sand, Coarse sand and Gravelly sand.

For the present study coarse sand media is assumed and accordingly the porosity is taken as 0.39.

9. Hydraulic conductivity = 480 m^3 / m^2 /day

   Table 3.4 of reference A recommends the hydraulic conductivity as 480 m^3 / m^2 /day corresponding to the porosity of 0.39 for coarse sand.

10. Size of gravel

    As per Table 3.4 of Reference A, the maximum 10% grain size is recommended as 2 mm. The same value is adopted and the sand shall be filtered accordingly.

11. Rate Constant at 20^o C

    As per Table 3.4 of Reference A, the rate constant at 20^o C is K_{20} = 1.35.

12. Check the product $K_s S$, which is a measure of hydraulic loading on the filter

    $K_s S$ = 480 x 1 / 100 = 4.8 m / day

    As per section 3.3.2 of Reference A, the product $K_s S \leq 8.6$ m / day

    In the present case, 4.8 m / day < 8.6 m / day and hence OK.

13. Rate Constant at design temperature

    The rate constant K_{20} should be adjusted to the design temperature by using the equation:

    $$K_T = K_{20} (1.1)^{T-20}$$

    $$K_T = 1.35 \times (1.1)^{(14.5-20)} = 0.79923$$

14. Required cross section area of the wetland is given by equation 3-8 of Reference A.

    $$A_c = \frac{Q}{K_s S}$$

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$A_c = \frac{250}{480 \ x \ 0.01}$ = 52.0833 m^2

15. Bed width is obtained by using equation 3-9 of Reference A

Bed width W = A_c / d = 52.0833 / 0.6 = 86.8056 m

16. Adopt a bed width of 87.00 m

17. Required surface area of the wetland is obtained using equation 3-7 of Reference A

$$A_s = \frac{[Q(lnC_o - lnC_e)]}{K_T dn}$$

$$A_s = \frac{[250(ln78 - ln30)]}{(0.79923)(0.6)(0.39)}$$

A_s = 1277.2798 m^2

18. Required length of filter bed

L = A_s / W = 1277.2798 / 86.8056 = 14.7143 l

19. Adopted length of the filter bed = 15 m

The length and width can be defined interchangeably, but the area should be maintained as the product of length and breadth. The adopted length / breadth is based upon the site conditions and the available size of the land parcel.

20. Hydraulic Retention time (HRT)

$$HRT = \frac{V_v}{Q} = \frac{LWdn}{Q}$$

$$HRT = \frac{14.7143 \ x \ 86.8056 \ x \ 0.6 \ x \ 0.39}{250} = 1.1955 \ days$$

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Table 2. 16 Summary of 0.25 MLD ABR + Constructed wetland

Unit Capacity (MLD)	Component	No.	L (m)	B (m)	D (m)
0.25	Anaerobic Settler	1	13.5	5.0	1.5
	Anaerobic Baffled Reactor	1	6.0	23.5	1.5
	Constructed Wet lands	1	87	15	0.6

S. No.	Component	No. of Units			Remarks
		1 MLD	2 MLD	5 MLD	
1	Anaerobic Settler	4	8	20	The 0.25 MLD module can be replicated as per site conditions
2	Anaerobic Baffled Reactor	4	8	20	
3	Constructed Wet lands	4	8	20	

In Figure 2.8, a sample arrangement of 0.25 MLD anaerobic settler, followed by Anaerobic Baffled Reactor and Constructed Wetlands is shown. The three reactors have to be placed in series. The sample arrangement is possible when a length of 106.5 is available for the reactors. If case the land dimensions donot pemit the placement of the reactors as shown, the reactors can be oriented / redimensioned to fit the available land dimension, as long as the arrangement is in series.

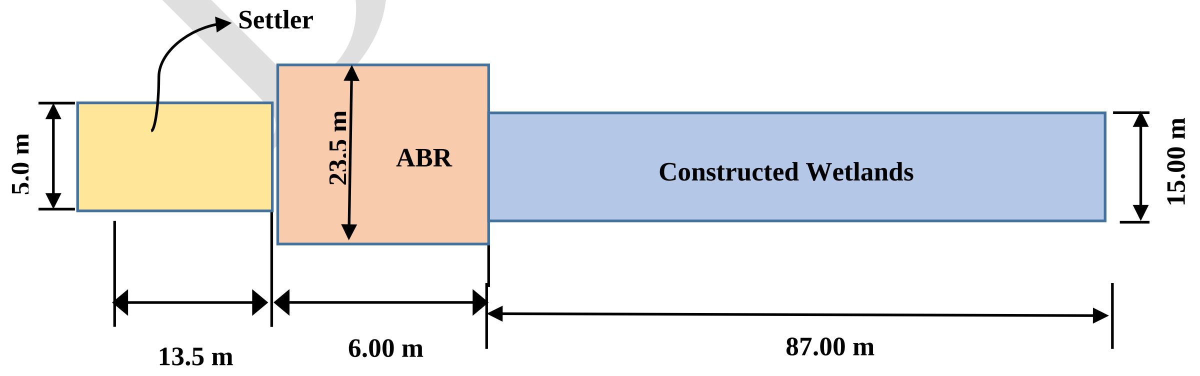
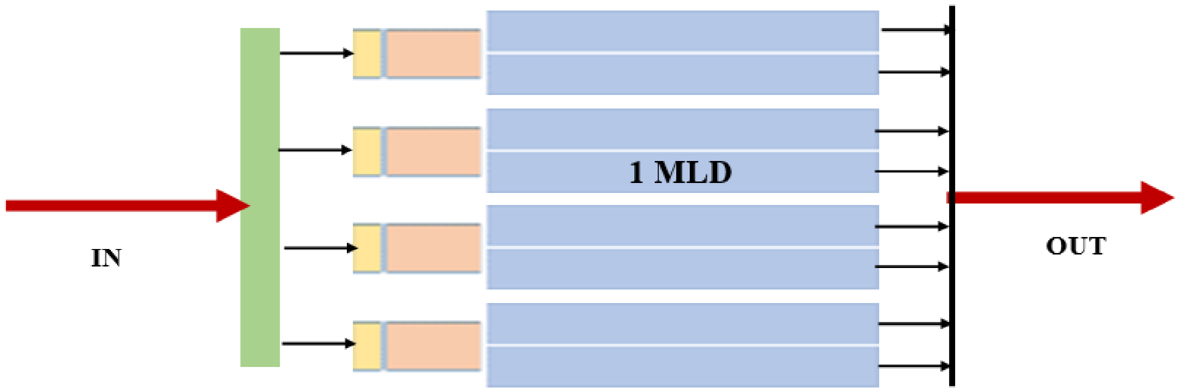


Figure 2.8 One of the possible arrangements for 0.25 MLD unit of ABR + CWL

The designed anaerobic settler, ABR and Constructed Wetland is for 0.25 MLD. This shall serve as the unit, which can be replicated in multiples of 0.25 MLD in order to construct plants of capacity more than 0.25 MLD. For example, 4 such units may be arranged in parallel as shown below in Figure 2.9 to construct a plant of capacity 1 MLD. The anaerobic settler, ABR and constructed wetland shall be in series in each unit, but the 4 units shall be in parallel.



For a plant of 4 MLD capacity, 16 units of 0.25 MLD shall be arraned in parallel. The arrangement shown is only for illustration purpose. The 16 units of 0.25 MLD may be arranged as permitted by the site conditions, as long as the anaerobic settler, ABR and constructed wetland of each 0.25 MLD unit is in series and the 16 units are in parallel. A suitable distribution and collection arrangement may be provided for the inflow and outflow.

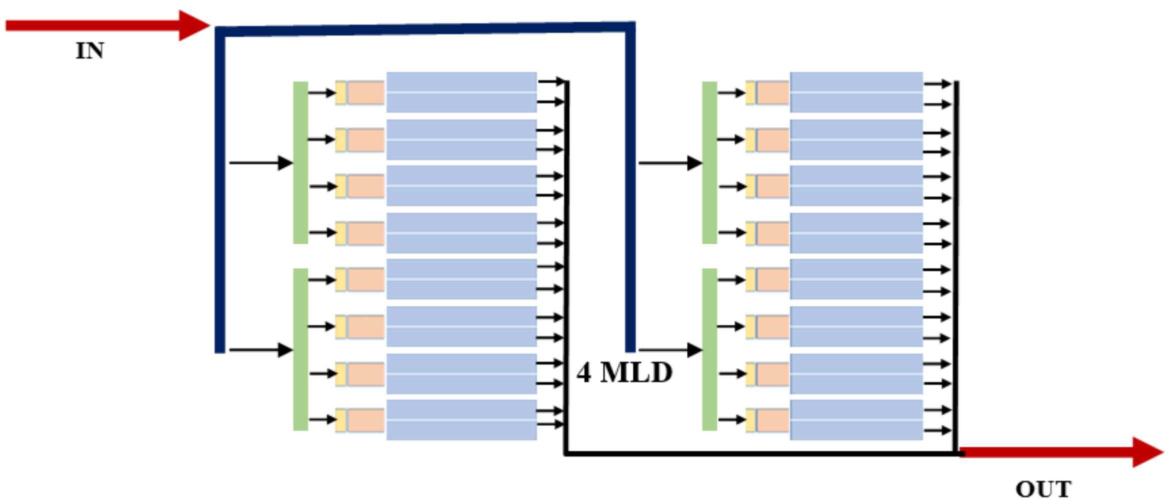


Figure 2.9 One of the possible arrangemnts for 1 and 4 MLD unit of ABR + CWL

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2.6 Design of UASB Reactor + Waste Stabilization Pond

The up-flow anaerobic sludge blanket reactor (UASBR) is a single tank anaerobic process which is known to achieve good removal of organic pollutants from the Used Water. It is advantageously used in countries with a warm climate throughout the year for both municipal as well as industrial wastewater. As with other anaerobic systems, the UASBR system also does not require energy for its operation and can be designed and constructed in a much simpler manner. A significant advantage is production of biogas, which can be collected and used for power generation or for cooking. The gas production is high in case of high strength wastewater such as industrial effluents.

The UASB reactor is reported to remove about 50 - 70% of the BOD and COD from the wastewater. Hence, the UASB effluent needs to be subjected to further aerobic treatment for reducing the BOD and nutrients to the required standard. Aerobic technologies like the activated sludge process and its variants are power intensive and hence will lead to higher O & M expenses. A more suitable technology would be constructed wetlands or stabilization ponds, in medium and small towns where land is available easily at a cheap rate.

2.6.1 Working of UASB Reactor

Post screening and grit removal, the waste water is taken down the UASB reactor of depth 4.5 – 5 m for BOD strength of 200 – 300 mg/l. The wastewater enters the reactor from the bottom and flows upward at a design velocity to the outlet which is located at the upper periphery of the reactor. A suspended sludge blanket filters and treats the wastewater as the wastewater flows through it. The usual hydraulic retention time (HRT) is 8 – 10 hours at average flow. The process is illustrated pictorially in Figure 2.10. The anaerobic unit does not need to be filled with any media as the up flowing sewage forms millions of microorganisms in the sludge which break down organic matter by anaerobic digestion, transforming it into biogas. Solids are also retained in the sludge blanket owing to a filtration effect. A high solid retention time of 30 – 50 days occurs in the reactor. The up-flow regime and the motion of the gas bubbles allow mixing without mechanical assistance. Baffles at the top of the reactor allow gases to escape and prevent an outflow of the sludge blanket.

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As in all anerobic treatments, some form of post-treatment is required after an UASB to remove residual COD / BOD, pathogens, and nutrients in order to meet the effluent discharge standards.

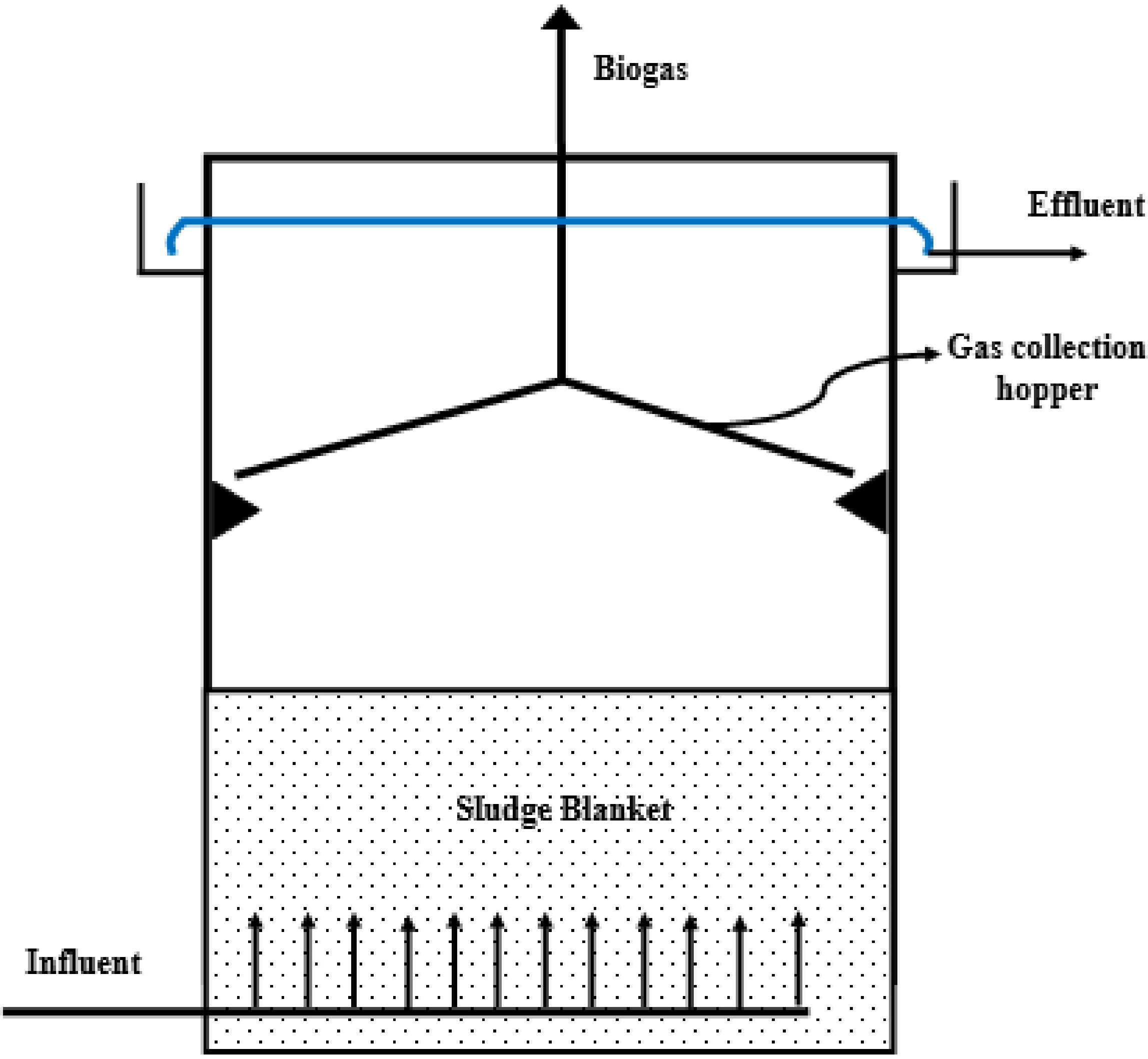

Figure 2. 7 Representation of the main components of UASB reactor

2.6.2 Post Treatments

There are several aerobic post treatment options for the effluent of the UASB reactor. In the simplest case, provision of cascades or detention in an algal oxidation pond would be adequate. Under aerobic biological treatment, activated sludge or extended aeration may be adopted. Natural systems like waste stabilization ponds or constructed wetlands etc. are also used.

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In the present study, waste stabilization ponds are proposed as post treatment. The design procedure has already been presented in Section 2.4.1. Two primary ponds are proposed in parallel followed by one secondary pond in series. Table 2.18 presents the design of UASB reactor followed by Table 2.19 for facultative pond.

Table 2. 17 Design calculations for the UASB Reactor of 1, 2 and 5 MLD capacity

DESIGN OF UASB REACTOR - 1 MLD, 2 MLD AND 5 MLD

S. No.	Design Item	1 MLD		2 MLD		5 MLD	
		Value	Units	Value	Units	Value	Units
	INPUT PARAMETERS						
1	Average flow	1.00	MLD	2.00	MLD	5.00	MLD
2	Average flow	1000.00	m^3/day	2000.00	m^3/day	5000.00	m^3/day
3	Average flow	41.67	m^3/hour	83.33	m^3/hour	208.33	m^3/hour
4	Influent BOD	250.00	mg/l	250.00	mg/l	250.00	mg/l
5	sBOD	90.00	mg/l	90.00	mg/l	90.00	mg/l
6	Influent COD	425.00	mg/l	425.00	mg/l	425.00	mg/l
7	Influent sCOD	115.00	mg/l	115.00	mg/l	115.00	mg/l
8	bCOD/BOD ratio	1.60		1.60		1.60	
9	rbCOD	70.00	mg/l	70.00	mg/l	70.00	mg/l
10	Influent SO_4	100.00	mg/l	100.00	mg/l	100.00	mg/l
11	Alkalinity	225.00	g / m^3 $CaCO_3$	225.00	g / m^3 $CaCO_3$	225.00	g / m^3 $CaCO_3$

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12	Influent TSS	375.00	mg/l	375.00	mg/l	375.00	mg/l
13	Influent VSS	262.50	mg/l	262.50	mg/l	262.50	mg/l
14	Temperature	25.00	ºC	25.00	ºC	25.00	ºC
	STANDARD DESIGN PARAMETERS						
15	Design Hydraulic Retention time (HRT) for average flow	6 to 12	hours	6.00	hours	6.00	hours
16	Design SRT	15 to 30	days	15 to 30	days	15 to 30	days
17	Efficiency of UASB Reactor	0.65	%	0.65	%	0.65	%
18	Organic Volumetric Loading	2.00	Kg COD /m^3.day	2.00	Kg COD/m^3.day	2.00	Kg COD /m^3.day
19	Average solids concentration in process volume, X_{VSS}	10000.00	mg / l	10000.00	mg/l	10000.00	mg/l
20	Upflow velocity through the sludge blanket	0.70	m / hoiur	0.70	m/hour	0.70	m/hour
21	Height of the UASB reactor	4.00	m	4.00	m	4.00	m
22	Velocity in the settling chamber	1.20	m / hour	1.20	m/hour	1.20	m/hour
23	Synthesis yield, Y_H	0.08	g VSS / g COD	0.08	g VSS / g COD	0.08	g VSS / g COD
24	Decay coefficient b_H	0.03	g /g.day	0.03	g/g.day	0.03	g/g.day
25	f_d	0.15		0.15		0.15	

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	EFFLUENT STANDARDS						
26	Effluent BOD	30.00	mg/l	30.00	mg/l	30.00	mg/l
27	Effluent COD	250.00	mg/l	250.00	mg/l	250.00	mg/l
28	Effluent Suspended Solids	100.00	mg/l	100.00	mg/l	100.00	mg/l
	REACTOR VOLUME CALCULATION						
29	Required Area of the reactor based on upflow velocity	59.52	m^2	119.05	m^2	297.62	m^2
30	Required volume of the reactor based on upflow velocity area	238.10	m^3	476.19	m^3	1190.48	m^3
31	Required volume of the reactor based on organic loading rate	212.50	m^3	425.00	m^3	1062.50	m^3
32	Adopted volume of the reactor	250.00	m^3	500.00	m^3	1250.00	m^3
33	Actual volumetric organic loading	1.70	Kg COD /m^3.day	1.70	Kg COD /m^3.day	1.70	Kg COD /m^3.day
34	HRT in days	0.25	days	0.25	m^3	0.25	m^3
35	HRT in hours	6.00	hours	6.00		6.00	
36	Reactor height to accommodate gas collection	2.50	m	2.50	m	2.50	m
37	Clear zone height	0.50	m	0.50	m	0.50	m
38	Total reactor height	7.00	m	7.00	m	7.00	m
39	L/B ratio for the reactor	2.00	ratio	2.00	ratio	2.00	ratio

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40	Provided area of the reactor	62.50	m^2	125.00	m^2	312.50	m^2
41	Length of the reactor	11.18	m	15.81	m	25.00	m
42	Width of the reactor	5.59	m	7.90	m	12.50	m
43	Adopted length of the reactor	11.20	m	15.90	m	25.00	m
44	Adopted width of the reactor	5.60	m	8.00	m	12.50	m
45	Area per feed inlet	2.00	m^2	2.00	m^2	2.00	m^2
46	No. of inlets required	31.25	Nos.	62.50	Nos.	156.25	Nos.
47	Adopted No. of inlets	32.00	Nos.	62.00	Nos.	155.00	Nos.
48	Area of settling chamber	34.72	m^2	69.44	m^2	173.61	m^2
	SRT CALCULATION						
49	Reduction of COD	276.25	mg/l	276.25	mg/l	276.25	mg/l
50	bCOD	400.00	mg/l	400.00	mg/l	400.00	mg/l
51	bpCOD/pCOD	0.82		0.82		0.82	
52	Effluent nbVSS concentration	45.72	mg/l	45.72	mg/l	45.72	mg/l
53	SRT	43.73	days	43.73	days	43.73	days
54	LHS of equation = $X_{VSS}(V)$	2500000.00	g/day	5000000.00	g/day	12500000.00	g/day
55	Component A of RHS of the equation	500302.67		1000605.34		2501513.35	

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56	Component B of RHS of the equation	1999697.32		3999394.65		9998486.64	
57	Equation set equal to zero in the Goal Seek of What If analysis	0.00		0.00		0.00	
	SLUDGE PRODUCTION						
58	Daily sludge production rate, $P_{X, vss}$	57.16	Kg VSS/day	114.33	Kg VSS/day	285.83	Kg VSS/day
59	Excess sludge daily waste volume	5.69	m^3/day	11.38	m^3/day	28.45	m^3/day
60	$P_{X, bio} = P_{X,VSS}$ - nbVSS(Q)	11440.10	g VSS/day	22880.20	g VSS/day	57200.51	g VSS/day
	METHANE PRODUCTION						
61	CH_4 COD per day	260005.05	CH4 COD per day	520010.11	CH4 COD per day	1300025.27	CH4 COD per day
62	Methane production rate at standard temperature of 0ºC	0.35	l methane / g COD	0.35	l methane / g COD	0.35	l methane / g COD
63	Methane production at standard temperature of 0ºC	91.00	m^3 Methane / day	182.00	m^3 Methane / day	455.01	m^3 Methane / day
64	Methane production rate at design temperature	99.33	m^3 Methane / day	198.66	m^3 Methane / day	496.65	m^3 Methane / day
65	Methane percentage in the gas produced	0.65	%	0.65	%	0.65	%
66	Total gas production rate	152.81	m^3 gas / day	305.63	m^3 gas / day	764.08	m^3 gas / day

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67	Energy content of methane gas	38846.00	KJ / m^3	38846.00	KJ / m^3	38846.00	KJ / m^3
68	Total energy in the methane produced	3535054.71	KJ / day	7070109.43	KJ / day	17675273.57	KJ / day
69	BOD_5 in the UASB reactor effluent	87.50	mg/l	87.50	mg/l	87.50	mg/l

Green cells: Input variable data to be provided Yellow cells: standard data from CPHEEO manual or other standards Red Cells: Design output

2.6.2.1 Design Steps of Up-flow Anaerobic Sludge Blanket (UASB) Reactor

REFERENCES FOLLOWED:

A. Wastewater Engineering Treatment and Reuse, Metcalf and Eddy, 5th edition, Mc Graw Hill Education, 2014.
B. Manual on Sewerage and Sewage Treatment Systems, third edition, CPHEEO, MoUD, 2013.
C. Wastewater Treatment for Pollution Control and Reuse, third edition, Soli J Arceivala and Shyam R Asolekar, Mc Graw Hill Education, 2007.

Input Data and Sewage Characteristics

1. STP capacity = average flow = 2 MLD

2. Average flow = $2.0 \times 10^6/10^3 = 2000\ m^3/day$

3. Average flow = $2000 / 24 = 83.3333\ m^3/hour$

4. Influent BOD = 250 mg/l

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As per clause 5.1.4.1 of CPHEEO Manual, '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. From Table 5.4, the recommended BOD is 250 mg/l.

5. sBOD = 90 mg/l

   The sBOD is a part of BOD_5 and represents that faction of BOD which is due to dissolved organic matter. The remaining BOD is due to particulate matter.

   The sBOD is not specified in the CPHEEO manual. It has to be determined by conducting laboratory test on sewage sample. In the present study, sBOD is taken as 90 mg / l. This value can be replaced with the actual sBOD as received from laboratory test report.

6. Influent COD = 425 mg/l

   The value of COD of influent sewage is taken from Table 5.4 of CPHEEO manual.

7. Influent soluble COD = sCOD = 115 mg/l

   The soluble COD is that component of COD which is in dissolved form and hence requires biological treatment for its removal. In contrast, insoluble COD is contributed by particulate matter and can be remove via filtration and / or settlement. The sCOD is not specified in the CPHEEO manual. It has to be determined by conducting laboratory test on sewage sample. In the present study, sCOD is taken as 115 mg / l. This value can be replaced with the actual sCOD as received from laboratory test report.

8. bCOD/BOD ratio = 1.6

   The biodegradable COD (bCOD) consists of the total fraction from COD that can be biodegraded by the heterotrophic microorganisms. Therefore it is the more proper parameter than total COD for estimating oxygen requirements.

   As per equation 8.8 in Metcalf and Eddy, the ratio of bCOD : BOD is 1.6.

9. Influent rbCOD = 70 mg/l

   The readily biodegradable COD (rbCOD) consists of small molecules that are directly available for biodegradation by heterotrophic microorganisms (volatile fatty acids,

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alcohols, amino-acids, simple sugars). The rbCOD is not specified in the CPHEEO manual. It has to be determined by conducting laboratory test on sewage sample. In the present study, rbCOD is taken as 90 mg / l. This value can be replaced with the actual rbCOD as received from laboratory test report.

10. Influent sulphate as SO_4^{-2} = 100 mg/l

In most of the literature, including CPHEEO manual, there is no mention about the SO_4^{-2} concentration in the municipal sewage. This is because sulphate is allowed up to 250 mg/l in potable water and there is no limit specified on sulfate in treated waste water for discharge in water bodies.

In the paper titled, ‘Effect of Sulfate Load on Sulfur Removal in Model Constructed Wetlands’ by Jiangxin Hou, Wenrui Guo, and Yue Wen, the sulfate concentration in the municipal sewage is said to generally vary from 40–200 mg/l. In the present study, the concentration of sulfate is taken as 100 mg/l with a clear understanding that the exact value will be obtained by carrying out test on the sewage.

11. Alkalinity = 225 mg/l

In the present study, the alkalinity of sewage is taken as 225 mg/l with a clear understanding that the exact value will be obtained by carrying out test on the sewage.

12. Influent TSS = 375 mg/l

As per Table 5.4 of CPHEEO manual, the influent total suspended solids is 375 mg/l in the absence of drain or outfall.

13. Influent VSS = 262.5 mg/l

As per Table 5.4 of CPHEEO manual, the influent volatile suspended solids is 262.5 mg/l in the absence of drain or outfall.

14. Temperature = 25^o C

In the present study, the plant is assumed to be located at a latitude of 21.14^o N (Central India) where the lowest winter temperature is obtained as 14.5^o C from the IMD

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records. Anaerobic treatment process is, in general, temperature sensitive and anaerobic processes are often maintained at temperature between 25^o C to 35^o C or higher. Heating of the influent and the reactor may be achieved by using a heater directly in the reactor vessel. In the present study, the temperature is found to be below 25^o C for 4 months of the year. Hence, heating may be resorted to during such times. The reference to heating can be found in section 10-5 of Metcalf and Eddy under the sub-heading of 'Temperature Management'

Standard Design Parameters

15. Hydraulic Retention Time (HRT) = 6 to 12 hours

As per clause 5.8.5.3.2 of CPHEEO manual, the HRT at average flow may be 6 to 12 hours for wastes containing suspended organic matter. As per Metcalf and Eddy, wastewaters with higher substrate concentration will require a longer hydraulic retention time for a given design Organic Loading Rate.As per equation 10-17 of Metcalf and Eddy, the HRT is given by:

HRT = S_o / OLR where S_o is the wastewater strength and OLR is the organic loading rate. Alternately, the HRT can also be obtained by using the equation:

HRT = V/Q, where V is the reactor volume and Q is the average flow.

16. Solids Retention Time (SRT) = 15 to 30 days

As per clause 5.8.5.3.1 of CPHEEO manual, the SRT ranges between 15 to 30 days.

As per Metcalf and Eddy, 'At 30^o C, SRT values greater than 20 days are needed for anaerobic processes for effective treatment performance. At lower temperatures much longer SRT values are needed. Recommended SRT values as a function of temperature for stable treatment of domestic wastewater by a UASB process are given in Table 10-12.

17. Efficiency of UASB reactor = 65%

As per Table 5.15 of CPHEEO manual, the efficiency ranges between 50 to 70%.

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Table 5.15 from CPHEEO Manual

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

18. Organic volumetric loading (OVL) = 2 Kg COD / m^3.day

As per Table 5.15 of CPHEEO manual the organic loading rate ranges between 1.0 to 2.0 Kg COD / m^3.day.

19. Average solids concentration in process volume, X_{VSS} = 10000 Kg / m^3

As per Table 10 – 6 of Metcalf and Eddy, the average solids concentration in the process volume varies from 10 to 30 Kg/m^3. Increasing the solids concentration will increase the SRT.

20. Up flow velocity through the sludge blanket = 0.7 m/hour

As per Table 10-17 of Metcalf and Eddy, the typical up flow velocity is 0.7 m/hour.

21. Height of the UASB reactor = 4.0 m

As per Table 10-17 of Metcalf and Eddy, the height of the UASB reactor for domestic wastewater ranges between 3 to 5 m.

22. Velocity in the settling chamber = 1.2 m / hour

As per Appendix 5-15 of CPHEEO manual, the settling chamber velocity is < 1.5 m/hour

23. Synthesis Yield, Y_H = 0.08 g VSS / g COD

Typical value for overall combined condition obtained from the data presented in Table 10-13 of Metcalf and Eddy.

24. Decay Coefficient b_H = 0.03

Typical value for overall combined condition obtained from the data presented in Table 10-13 of Metcalf and Eddy.

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25. Biomass decay coefficient, f_d for COD oxidation = 0.15

Value obtained from the data presented in Table 8-14 of Metcalf and Eddy.

Effluent Standards

26. BOD = 30 mg/l

27. COD = 250 mg/l

28. TSS = 100 mg/l

With reference to Table 5.3 of the CPHEEO manual, the permitted level of BOD_5, COD and TSS for discharge into inland surface water bodies is considered. The table is based on General standards for Discharge of Environmental Pollutants, Part A: Effluents. The present design is executed in order to meet the above-mentioned limits.

Reactor Volume Calculation

29. Required area of the reactor based on up-flow velocity is

$A = Q/v$ Equation 10-18 of Metcalf and Eddy

Where

A = reactor cross-section area, m^2

Q = influent flow rate, m^3/h

v = maximum design up-flow velocity, m/h

$A = 2000 / (0.7 \times 24) = 119.0476\ m^2$

30. Required volume of the reactor based on up-flow velocity = A x height

$V_v = 119.0476 \times 4 = 476.1905\ m^3$

31. Required volume of reactor based on organic loading rate:

$V_{OLR} = Q\ S_o / OLR$ Equation 10-20 of Metcalf and Eddy.

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= 2000 x 425 / (2 x 1000) = 425 m^3

32. Adopted volume of the reactor = Maximum of volumes from up-flow velocity and OLR = Max[V_v, V_{OLR}] = Max[476.1905, 425] = 476.1905 = say 500 m^3
Note: Choose a value for volume such that the HRT is between 6 to 12 hours.

33. Actual volumetric loading = OLR = 2 x 425 / 500 = 1.7000 Kg COD / m^3.day < 2 Kg COD / m^3.day. Hence OK.

34. HRT = Reactor Volume / Average flow = 500 / 2000 = 0.25 days

35. HRT = 0.25 x 24 = 6.00 hours
Which is as per clause 5.8.5.3.2 of CPHEEO Manual. Hence OK.

36. Reactor height to accommodate gas collection = 2.5 m
In accordance to Table 10-15 of Metcalf and Eddy.

37. Clear zone height = 0.5 m
In accordance to the recommended value in Metcalf and Eddy.

38. Total reactor height = 4 + 2.5 + 0.5 = 7.0 m.

39. L/B ratio = 2.0
The ratio is assumed as 2.0. There is no recommendation on L/B ratio in CPHEEO manual.

40. Provided area of the reactor = Volume / Height = 500 / 4 = 125.0000 m^2

41. Provide rectangular section for the reactor.
Length of the reactor = L = $\sqrt{2\ (A)}$ = $\sqrt{2\ (125)}$ = 15.8114 m

42. Width of the reactor = B = L/2 = 11.1803 / 2 = 7.9057 m

43. Adopted length of the reactor by rounding off = 15.9 m

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44. Adopted width of the reactor by rounding off = 8.0 m

45. Area per feed inlet = 2 m^2

The value varies between 1 to 2 m^2 for OLR between 1 to 2 Kg COD / m^3.day as per Table 10-16 of Metcalf and Eddy

46. Number of inlets required = Reactor Area / Area per inlet = 125 / 2 = 62.50 Nos.

47. Adopted number of inlets = 62 Nos.

48. Area of settling chamber = Q / (24 x velocity in settling chamber) = 2000 / (24 x 1.5) = 69.4444 m^2

SRT Calculations

49. Reduction of COD = (S_o – S) = 0.65 x S_o = 0.65 x 425 = 276.25 mg / l

Efficiency as per Table 5.15 of CPHEEO manual = 50 – 75%, say 65%

50. bCOD = $(bCOD/BOD)_{Ratio}$ x BOD = 1.6 x 250 = 400 mg/l

51. $\frac{bpCOD}{pCOD} = \frac{bCOD}{BOD} x \frac{(BOD-sBOD)}{(COD-sCOD)} = 1.6x\frac{(250-90)}{(425-115)} = 0.8258$

52. nbVSS concentration in the effluent = (1 - $\frac{bpCOD}{pCOD}$) x VSS

= (1-0.8258) x 262.5 = 45.7258 mg/l

53. Determination of SRT is done by using equations 7-56, 8-20 of Metcalf and Eddy, 5th edition.

$$Mass\ of\ MLVSS = X_{VSS}(V) = (P_{X.VSS})SRT\ \ Eq.7-56\ of\ Metcalf\ and\ Eddy$$

$$(P_{X.VSS}) = \frac{QY_H(S_o - S)}{1 + b_H(SRT)} + \frac{f_d b_H QY_H(S_o - S)SRT}{1 + b_H(SRT)} + Q(nbVSS)\ \ Equation\ 8-20$$

Substitute Equation 8-20 into Equation 7.56 to obtain

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$$X_{VSS}(V) = \frac{QY_H(S_o - S)SRT(1 + f_d b_H SRT)}{1 + b_H(SRT)} + Q(nbVSS)SRT$$

SRT = 1.0 days

Assume a starting value for SRT and solve equation 8.16 by trial and error using the 'Goal Seek' function in Data Tab of MS Excel.

54. $X_{VSS}(V) = 10000\ x\ 500 = 5000000\,\frac{g}{day} = 5000\,\frac{Kg}{day}$

55. Component A of Equation

$$\frac{QY_H(S_o - S)SRT(1 + f_d b_H SRT)}{1 + b_H(SRT)}$$

$$= \frac{2000(0.08)(276.25)(SRT)(1 + (0.15)(0.03)(SRT))}{1 + 0.03(SRT)}$$

$$= \frac{44200(SRT)(1 + 0.0045(SRT))}{1 + 0.03(SRT)}$$

56. Component B of Equation = $Q(nbVSS)SRT$ = 2000 x 45.7258 x SRT

= 91451.6129 (SRT)

57. 5000000 = $= \frac{44200(SRT)(1+0.0045(SRT))}{1+0.03(SRT)} + 91451.6129$ (SRT)

Solve for SRT by trial and error to get

SRT = 43.7324 days.

As per Table 10-12 of Metcalf and Eddy, the SRT for stable operation for treatment of domestic wastewater is up to 60 days. Hence the obtained value of 43.7324 days is satisfactory.

Sludge Production

58. $P_{X,VSS} = X_{VSS}$ (V)/SRT … Equation 7-56 of Metcalf and Eddy

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= 10000 x 500 / (43.7324 x 1000) = 114.3318 Kg VSS / day

59. Q_w = Excess sludge daily waste volume is obtained using modified form of equation 8-27 of Metcalf and Eddy.

$P_{X,VSS} = Q X_e + X Q_w$ modified form of equation 8-27 of Metcalf and Eddy

Where:

X_e = effluent TSS concentration, mg / l

$Q_w = (P_{X,VSS} - Q X_e) / X$ = (114.3318 x 1000 – 2000 x 262.5/1000) / 10000

= 11.3807 m^3 / day

60. Compute the biomass production (VSS) using the equation 8-20 of Metcalf and Eddy

$P_{X,\,bio} = P_{X,VSS}$ - nbVSS(Q) Equation 8-20 of Metcalf and Eddy

$P_{X,\,bio}$ = 114.3318 x 1000 – 45.7258 x 2000

= 22880.2057 g VSS / day

Determine the production of methane gas and energy content

61. Determine the methane gas production rate by COD balance.

The COD of cell tissue is 1.42 g COD / g VSS based upon equation 7.5 of Metcalf and Eddy.

COD removal = methane COD + biomass COD

Methane COD = COD removed – biomass COD

= $Q(S_o – S)$ – 1.42 $P_{X,,bio}$

= 2000 x 276.25 – 1.42 x 22880.2057 = 520010.1079 methane COD / day

62. Methane production rate at standard temperature of 0^o C = 0.35 m^3 methane / Kg COD removed. This is based on Table 10-13 of Metcalf and Eddy.

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63. Methane production at standard temperature of 0^o C = Methane COD x Methane production rate

= 520010.1079 x 0.35 = 182.0035 m^3 methane / day

64. Methane production at design temperature of 25^o C

= 182.0035 x (273.15 + 25)/273.15 = 198.6614 m^3 methane / day

The production rate varies linearly as the ratio of absolute design temperature to standard absolute temperature.

65. Percentage of methane in the gas produced in the process = 65%

The percentage is based on Table 10-13 of Metcalf and Eddy.

66. Total gas production = methane produced / percentage of methane in gas

=198.6614 / 0.65 = 305.6329 m^3 gas / day

67. Energy content of methane gas = 38846 KJ / m^3

The value if based on Table 10-13 of Metcalf and Eddy.

68. Total energy possessed by the volume of methane gas produced per day

= 182.0035 x 38846 = 7070109.428 KJ / day.

Effluent BOD

69. BOD_5 in the effluent from the UASB reactor, assuming 65% efficiency of treatment

= 250 x (1 – 0.65) = 87.5 mg/l.

The facultative pond is designed to reduce the BOD from 87.5 mg/l to 30 mg/l.

Table 2. 18 Design calculations for the facultative Ponds of 1, 2 and 5 MLD capacity

DESIGN OF FACULTATIVE (WASTE STABILIZATION) POND OF 1, 2 AND 5 MLD CAPACITY

S. No.	Design parameter	Design output

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1	Capacity of the STP	1	MLD		2	MLD		5	MLD
2	Capacity of the STP	1000	m^3/day		2000	m^3/day		5000	m^3/day
3	Capacity of the STP	0.011 57	m^3/s		0.0231 5	m^3/s		0.0578 7	m^3/s
4	Per capita supply	135	LPCD		135	LPCD		135	LPCD
5	Influent BOD_5 (As per Table 5.4 of CPHEEO Manual - 2013)	27	g/capita per day		27	g/capita per day		27	g/capita per day
6	Influent BOD_5 for UASB reactor	250	mg/l		250	mg/l		250	mg/l
7	Influent BOD, for Facuiltative pond	87.50	mg/l		87.50	mg/l		87.30	mg/l
8	Effluent BOD_5 (As per Schedule VI of environment (protection) third Amendment Rules, 1993 - for Inland surface water)	30	mg/l		30	mg/l		30	mg/l
9	Latitude (Chosen for sample design corresponding to a centrally located city in the country.	21.14	degree		21.14	degree		21.14	degree
10	Average ambient temperature in December (https://nagpur.gov.in/geography-climate/)	14.5	K		14.5	K		14.5	K
11	Altitude of the city	310	m		310	m		310	m
12	Permissible pond loading rate according to latitude as per Table 1 of IS : 5611 - 1987 or Table 5.14 of CPHEEO manual	242.8 8	Kg/ha/day		242.88	Kg/ha/day		242.88	Kg/ha/day
13	Correction of pond loading rate for altitude = (1+0.003 x altitude in m)	1.93			1.93			1.93	

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14	Corrected pond loading rate as per latitude and elevation	125.84	Kg/ha/day	125.84	Kg/ha/day	125.84	Kg/ha/day
15	Pond loading rate according to temperature = 20T-120	170	Kg/ha/day	170	Kg/ha/day	170	Kg/ha/day
16	Adopt the average and the design pond loading rate	147.92	Kg/ha/day	147.92	Kg/ha/day	147.92	Kg/ha/day
17	Design pond loading rate, say	148	Kg/ha/day	148	Kg/ha/day	148	Kg/ha/day
18	BOD loading rate from the town	87.5	Kg/ha/day	175	Kg/ha/day	436.5	Kg/ha/day
19	Pond area required	0.59	ha	1.18	ha	2.95	ha
20	Depth of the pond	1.5	m	1.5	m	1.5	m
21	Pond volume	8868.24	m^3	17736.49	m^3	44239.86	m^3
22	Pond detention time	8.87	days	8.87	days	8.85	days
23	No. of ponds in parallel	2		2		2	
24	No. of ponds in series	1		1		1	
25	Total number of ponds	3		3		3	
26	BOD reaction rate K_l	0.2		0.2		0.2	
27	Percentage reduction of BOD required	0.66		0.66		0.66	
28	Detention time required for plug flow condition	5.35	days	5.35	days	5.34	days
29	Detention time required for completely mixed flow	7.25	days	7.25	days	7.23	days

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30	Population equivalent	7407			14815			37037	
31	Area per person (net)	0.798	m^2		0.798	m^2		0.796	m^2
32	Rate of sludge accumulation	0.07	m^3/person/ day		0.07	m^3/person/ day		0.07	m^3/person/ day
33	Sludge volume in 1 year	518.52	m^3		1037.03	m^3		2592.59	m^3
34	Depth in the first cell to accommodate the sludge	2	m		2	m		2	m
35	No. of first cells allocated for sludge accumulation	2	No.		2	No.		2	No.
36	Depth of sludge in the first 2 cells	0.5	m		0.5	m		0.5	m
37	Volume of sludge in the first 2 cell(s)	1970.72	m^3		3941.44	m^3		9831.08	m^3
38	Duration of sludge cleaning	3.80	years		3.80	years		3.79	years
39	Area of each pond	0.20	ha		0.39	ha		0.98	ha
40	L/B raio	4	dimensionless		4	dimensionless		4	dimensionless
41	Length L	88.78	m		125.56	m		198.30	m
42	Breadth B	22.20	m		31.39	m		49.58	m
43	First 2 Tank L x B X D (Mid-Depth)	89 x 22 x 2	m		126 x 32 x 2.0	m		200 x 50 x 2	m
44	Subsequent Tanks L x B X D (Mid-Depth)	89 x 22 x 1.5	m		126 x 32 x 1.5	m		200 x 50 x 1.5	m
45	Length at mid-depth of primary and secondary tanks	89	m		126	m		200	m

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46	Width at mid-depth of primary and secondary tanks	22	m		32	m		50	m
47	Side slope of pond is 1V :2.0 H	2	m		2	m		2	m
48	Top of embankment above mid depth in primary tank	2	m		2	m		2	m
49	Tank Bottom below mid-depth in primary tank	1	m		1	m		1	m
50	Top of embankment above mid depth in secondary tank	1.75	m		1.75	m		1.75	m
51	Tank Bottom below mid-depth in Secondary tank	0.75	m		0.75	m		0.75	m
52	Top length in primary tank	97	m		134	m		208	m
53	Top width in primary tank	30	m		40	m		58	m
54	Bottom length in primary tank	85	m		122	m		196	m
55	Bottom width in primary tank	18	m		28	m		46	m
56	Top length in secondary tank	96	m		133	m		207	m
57	Top width in secondary tank	29	m		39	m		57	m
58	Bottom length in secondary tank	86	m		123	m		197	m
59	Bottom width in secondary tank	19	m		29	m		47	m

Green cells: Input variable data to be provided Yellow cells: standard data from CPHEEO manual or other standards Red Cells: Design output

Summary of the UASB + Waste Stabilization Pond components is presented in Table 2.20.

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Table 2. 19 Summary of the design of UASB + post treatment through WSP

Component	Capacity		
	1 MLD`	2 MLD	5 MLD
No. of UASB reactors	1	1	1
L (m)	11.5	15.9	25
B (m)	5.6	8.2	12.5
D (m)	7.0	7.0	7.0
No. of WSPs	3 - 2 in parallel +1 in series	3 - 2 in parallel +1 in series	3 - 2 in parallel +1 in series
	Primary Pond (2 Nos. in parallel)		
L (m) (Top/Mid/Bottom)	97/89/85	134/126/122	208/200/196
B (m) (Top/Mid/Bottom)	30/22/18	40/32/28	58/50/46
D (m)	2.0	2.0	2.0
	Secondary Pond (1 No. in series)		
L (m) (Top/Mid/Bottom)	96/89/86	133/126/123	207/200/197
B (m) (Top/Mid/Bottom)	29/22/19	39/32/29	57/50/47
D (m)	1.5	1.5	1.5

2.7 Design of UASB Reactor + Activated Sludge Process (ASP)

Description of UASB Reactor has already been furnished in sections 2.6 and 2.6.1. The UASB reactor is reported to remove about 60% of the BOD from the wastewater. This necessitates adopting a suitable post treatment process to make the effluent meet the disposal standards.

The post treatment process is an aerobic process which helps in removal of residual BOD and COD, reduction of nutrients and pathogens. In Section 2.6, waste stabilization ponds were used as post treatment units. In this section, activated sludge process is proposed for aerobic post treatment.

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2.7.1 Activated Sludge Process

The activated sludge process is perhaps one of the most extensively used technologies for the treatment of municipal sewage. The conventional activated sludge process for treatment of municipal and industrial wastewater was developed between 1912 – 1914. Since then there have been many variations in the process leading to different designs. However, in general all the Activated Sludge Processes have three main components; a bio reactor consisting of aeration tank, a secondary settling or sedimentation tank and a system for partially transferring the activated sludge from settling tank to the aeration tank, called the return activated sludge.

As part of the treatment process, air is injected into raw unsettled sewage in the aeration tank. The supply of air (oxygen) may be accomplished by using surface aerators or diffusers at the bottom of the tank. Oxygen is used to establish and regulate aerobic conditions and to suspend sludge.

The following are some of the benefits of ASP: Reduction of sludge quantity, reseeding of the bacteria amongst themselves, Dependability, ease of understanding and operation, odorless, and economical. The land requirement is substantially reduced in comparison to the natural systems like waste stabilization ponds and constructed wetlands. Due to this, ASP has found wide acceptance in medium and large towns where land is not easily available for construction of the treatment plant.

2.7.2 Working of ASP

The activated sludge process is conventional biological treatment of wastewater under aerobic conditions. The ASP consists of an aeration tank, where organic matter is stabilized by the action of bacteria under aeration and a secondary sedimentation tank (SST), where the biological cell mass is separated from the effluent of aeration tank and the settle sludge is recycled partly to the aeration tank and remaining is wasted. The aeration is achieved by use of submerged diffusers or mechanical surface aerators. If diffusers are used to supply air, a spiral flow patter is obtained in the tank. In contrast, a completely mixed reactor can also be designed. The flow chart of the process is shown in Figure 2.11.

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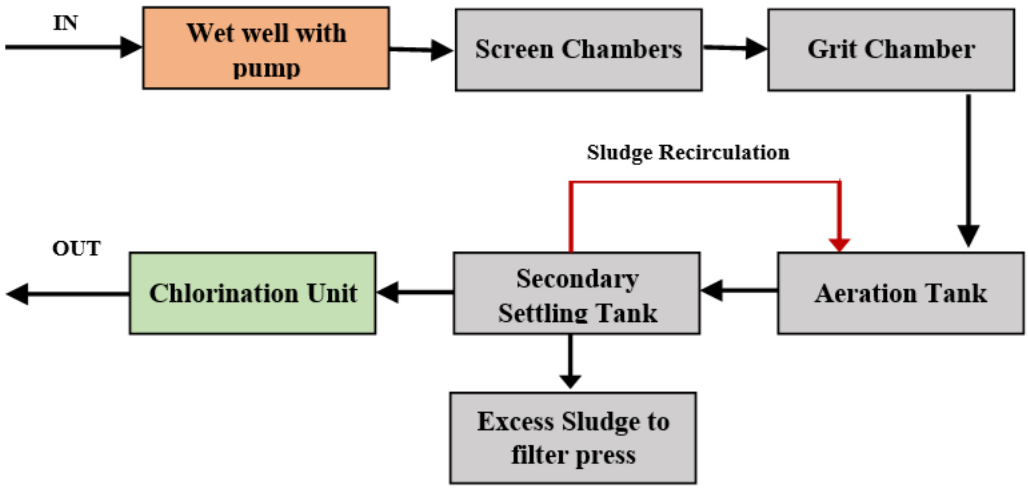

Figure 2. 8 Flowchart showing the conceptualization of Activated Sludge Process

In conventional ASP the flow model in aeration tank is plug flow type, with a volumetric loading rate of 0.3 to 0.6 Kg BOD / m3/day. The MLSS is maintained between 1500 -3000 mg/l. The mean cell residence time of 5 – 15 days is maintained. The hydraulic retention time (HRT) is maintained between 4 – 8 hours. Higher HRT may be warranted in stronger wastewaters from industrial processes. The sludge recirculation ratio is generally maintained between 0.25 and 0.50. The F / M ratio is usually maintained between 0.2 and 0.4 for conventional ASPs and between 0.3 and 0.6 for completely mixed ASP.

Post-secondary settling, the effluent is subjected to disinfection using chlorine gas prior to release in the environment.

2.7.3 Design Calculations of UASBR + ASP

The design calculations for UASB reactor of capacity 1 MLD, 2 MLD and 5 MLD has already been presented in Table 2.18. The BOD load on the ASP reactor is 87.5 mg/l, which is the BOD of the effluent from the UASB reactor. The flowchart of the proposed treatment scheme is shown in Figure 2.12.

As can be seen in the flow chart, the excess sludge from UASB reactor, aeration tank and secondary settling tank is sent to the filter press for processing and disposal.

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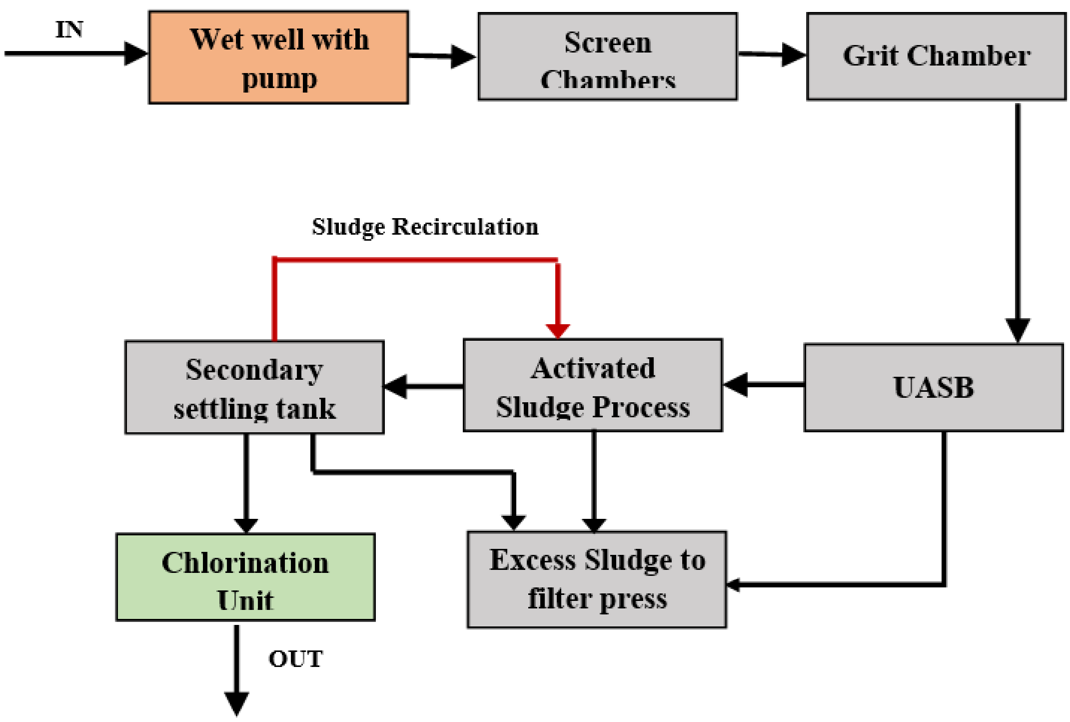

Figure 2. 12 Flowchart showing the conceptualization of Activated Sludge Process

The design calculations are presented in Table 2.22 and summary of the combined design of UASBR and ASP reactor is presented in Table 2.23. The aeration tank is designed as a conventional plug flow reactor with the standard volumetric loading, F:M ratio and HRT as given in Table 2.21, which contains typical values of design parameters for ASP are given in Table 8-16 of 'Wastewater Engineering, Treatment and Reuse by Metcalf and Eddy, 4th edition, 2003. Extracts from the table are presented below in Table 2.21.

Table 2. 20 Typical range of activated sludge design parameters

Technology	SRT (days)	F:M (Kg BOD / Kg MLVSS.day)	Volumetric Loading (Kg BOD / m^3.day)	MLSS (mg/l)	HRT (hours)	RAS % of influent
ASP	3 – 15	0.20 – 0.60	0.30 – 1.60	1500 - 4000	3 – 6	25-100

Extended aeration	20 - 40	0.04 – 0.10	0.10 – 0.30	2000 - 5000	20 - 30	50-150

Table 2. 21 Design calculations of 2 MLD ASP reactor

DESIGN OF 1 MLD, 2 MLD AND 5 MLD STP BASED ON ACTIVATED SLUDGE PROCESS									
S. No.	**Item**	**1 MLD**			**2 MLD**			**5 MLD**	
		Value	**Unit**		**Value**	**Unit**		**Value**	**Unit**
	INPUT DATA AND SEWAGE CHARACTERISTICS								
1	Average design flow into the STP	1.00	MLD		2.00	MLD		5.00	MLD
2	Average design flow into the STP	1000.00	m^3/day		2000.00	m^3/day		5000.00	m^3/day
3	Average design flow into the STP	41.67	m^3/hour		83.33	m^3/hour		208.33	m^3/hour
4	Average design flow into the STP	0.0116	m^3/s		0.0231	m^3/s		0.0579	m^3/s
5	Elevation of site above MSL	310.00	m		310.00	m		310.00	m
6	Operating temperature	14.50	ºC		14.50	ºC		14.50	ºC
7	Influent wastewater BOD	87.50	mg/l		87.50	mg/l		87.50	mg/l
	EFFLUENT STANDARDS								
8	Effluent BOD	30.00	mg/l		30.00	mg/l		30.00	mg/l
9	Effluent COD	250.00	mg/l		250.00	mg/l		250.00	mg/l
10	Effluent TSS	100.00	mg/l		100.00	mg/l		100.00	mg/l

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	DESIGN PARAMETERS AND ASSUMPTIONS							
11	Thickener overflow return as fraction of plant flow	0.15	Dimensionless		0.15	Dimensionless		0.15	Dimensionless
12	Thickener overflow return	0.15	MLD		0.30	MLD		0.75	MLD
13	Thickener overflow return BOD	500.00	mg/l		500.00	mg/l		500.00	mg/l
14	Centrate from sludge dewatering as fraction of plant flow	0.006			0.006			0.006	
15	Centrate from sludge dewatering return	0.006	MLD		0.012	MLD		0.030	MLD
16	Centrate from sludge dewatering return BOD	380.00	mg/l		380.00	mg/l		380.00	mg/l
17	Influent BOD to aeration tank	142.54	mg/l		142.54	mg/l		142.54	mg/l
18	Weighted BOD to be removed in the aeration tank	112.54	mg/l		112.54	mg/l		112.54	mg/l
19	MLSS	3000.00	mg/l		3000.00	mg/l		3000.00	mg/l
20	F : M Ratio	0.35	1/day		0.35	1/day		0.35	1/day
21	Total volatile fraction (MLVSS) of MLSS	0.80			0.80			0.80	
	DETERMINATION OF REACTOR VOLUME								
	Required Reactor Volume based on F:M ratio								
22	F	130.10	Kg/day		260.20	Kg/day		650.50	Kg/day
23	M	371.71	Kg		743.43	Kg		1858.57	Kg
24	Aeration tank volume calculated from F/M using Equation 5.27 of CPHEEO Manual	154.88	m^3		309.76	m^3		774.40	m^3
	Required Reactor Volume based on SRT								

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25	Mean Cell Residence Time or SRT, θ_c as per Figure 5.38 of CPHEEO Manual	8.00	days		8.00	days		8.00	days
26	Constant Y	0.50			0.50			0.50	
27	constant Kd	0.06	1/day		0.06	1/day		0.06	1/day
28	Aeration tank volume calculated from SRT using Equations 5.24 and 5.25 of CPHEEO Manual	101.39	m^3		202.78	m^3		506.95	m^3
	Required Reactor Volume Based on HRT								
29	HRT for average flow as per Table 5.9 of CPHEEO manual	5.00	hours		5.00	hours		5.00	hours
30	Aeration tank volume calculated from HRT	208.33	m^3		416.67	m^3		1041.67	m^3
31	Maximum of the three volumes of aeration tank, cum	208.33	m^3		416.67	m^3		1041.67	m^3
32	Volumetric BOD loading rate	0.54			0.540			0.54	
	SIZING OF THE REACTOR								
33	Depth of liquid in the reactor tank	5.50	m		5.50	m		5.50	m
34	L/B ratio	1.00	m		1.00	m		1.00	m
35	Length of the tank	6.15	m		8.70	m		13.76	m
36	Width of the tank	6.15	m		8.70	m		13.76	m
37	Adopted length of the tank	6.25	m		8.75	m		14.00	m
38	Adopted width of the tank	6.25	m		8.75	m		14.00	m
	DETERMINATION OF AERATION REQUIREMENT								
39	BOD removed in aeration tank	130.10	Kg/day		260.20	Kg/day		650.50	Kg/day

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40	Oxygen requirement at Kg oxygen / Kg of BOD removed	0.90	Kg O_2/Kg BOD		0.90	Kg O_2/Kg BOD		0.90	Kg O_2/Kg BOD
41	Kg of Oxygen needed per day	117.09	Kg O_2 per day		234.18	Kg O_2 per day		585.45	Kg O_2 per day
42	Residual D. O. in aeration tank	2.00	mg/l		2.00	mg/l		2.00	mg/l
	CAPACITY OF SURFACE AERATORS								
43	α, ratio of oxygen uptake rate of sewage to that of clean tap water at 20ºC	0.83			0.83			0.83	
44	β, multiplying factor for dissolved oxygen saturation for sewage at operating temperature	0.95			0.95			0.95	
45	D O at operating temperature	10.37	mg/l		10.37	mg/l		10.37	mg/l
46	D O at operating elevation	10.01	mg/l		10.01	mg/l		10.01	mg/l
47	Oxygen tension, mg/l	7.51	mg/l		7.51	mg/l		7.51	mg/l
48	Oxygen gradient, mg/l	0.82	mg/l		0.82	mg/l		0.82	mg/l
49	Temperature difference	-5.50	ºC		-5.50	ºC		-5.50	ºC
50	Temperature Co-efficient	1.02			1.02			1.02	
51	Temperature correction factor	0.88			00.88			0.88	
52	Conversion factor to standard conditions	0.596			0.596			0.596	
53	Oxygen needed under standard conditions, kg / day	196.33	Kg/day		392.64	Kg/day		981.61	Kg/day
54	Provide factor of safety for intangibles	1.10			1.10			1.10	
55	Oxygen needed after factor of safety	215.95	Kg/day		431.91	Kg/day		1079.77	Kg/day
56	Oxygen transfer capacity of aerator	1.80	Kg/KWh		1.80	Kg/KWh		1.80	Kg/KWh

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57	Required Aerator Capacity	5.00	KW		10.00	KW		25.00	KW
	CAPACITY OF DIFFUSED AERATORS								
58	Standard temperature	20.0	ºC		20.0	ºC		20.0	ºC
59	Density of air at operating temperature	1.24	Kg/m^3		1.24	Kg/m^3		1.24	Kg/m^3
60	Density of air at 20ºC temperature	1.23	Kg/m^3		1.23	Kg/m^3		1.23	Kg/m^3
61	Content of oxygen in air	0.23			0.23			0.23	
62	Kg of oxygen needed for residual D O per day	2.00	Kg / 2 mg/l O_2		4.00	Kg / 2 mg/l O_2		10.00	Kg / 2 mg/l O_2
63	Total kg of oxygen needed per day	119.09	Kg/day		238.18	Kg/day		595.45	Kg/day
64	cum of air needed per day	412.55	m^3/day		825.11	m^3/day		2062.77	m^3/day
65	Transfer efficiency of diffuser system per m depth	0.05			0.05			0.05	
66	Transfer efficiency at design depth	0.28			0.28			0.28	
67	diffuser fouling factor per year	0.04	1/year		0.04	1/year		0.04	1/year
68	Diffuser life cycle	3.00	years		3.00	years		3.00	years
69	Diffuser fouling factor for its life cycle	1.12			1.12			1.12	
70	Provide factor of safety for intangibles	1.10			1.10			1.10	
71	Air needed for oxygenation	1856.27	m^3/day		3712.53	m^3/day		9281.33	m^3/day
72	Air needed for oxygenation in cum / hour	77.34	m^3/hour		154.69	m^3/hour		386.72	m^3/hour
73	Air mixing criteria cum /minute / 1000 cum of tank	16.00	m3/hour /1000 cum		16.00	m3/hour/1000 cum		16.00	m3/hour /1000 cum

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74	Air needed for mixing as per manual cum / hr	200.00	m^3/hour		400.00	m^3/hour		1000.00	m^3/hour
75	Minimum air needed for mixing as per manual	2.70	m^3/hour/m^2		2.70	m^3/hour/m^2		2.70	m^3/hour/m^2
76	Surface area of aeration tank	37.88	m^2		75.76	m^2		189.39	m^2
77	Air needed for mixing as per US EPA guidelines	102.27	m^3/hour		204.54	m^3/hour		511.36	m^3/hour
78	Adopted value of air needed for mixing	200.00	m^3/hour		400.00	m^3/hour		1000.00	m^3/hour
79	Air needed as under standard conditions	335.33	m^3/hour		670.67	m^3/hour		1676.68	m^3/hour
80	Friction and other losses as fraction of depth	0.20			0.20			0.20	
81	Liquid depth as water column for air pressure	6.60	m		6.60	m		6.60	m
82	KW of needed compressor at 1400 rpm	19.44	KW		26.94	KW		49.46	KW
	SLUDGE PRODUCTION								
83	Yobs=Y/(1+Kd*θc)	0.34			0.34			0.34	
84	Excess Sludge mass wasted Kg/day = A	38.02	Kg/day		76.04	Kg/day		190.11	Kg/day
85	Kg of excess sludge / Kg of BOD removed	0.40			0.40			0.40	
86	Kg of excess sludge from thumb rule per day	45.02	Kg/day		90.03	Kg/day		225.09	Kg/day
87	Excess sludge as higher of the two values	45.02	Kg/day		90.03	Kg/day		225.09	Kg/day
88	Concentration factor for MLSS in return / excess sludge	3.30			3.30			3.30	
89	Return / excess sludge MLSS concentration	9900.00	mg/l		9900.00	mg/l		9900.00	mg/l

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		Variable input							
		Input from CPHEEO manual or other standards							
		Output							
90	Cells in aeration tank based on MLSS	625.00	Kg		1250.00	Kg		3125.00	Kg
91	Cells wasting from system	45.02	Kg/day		90.03	Kg/day		225.09	Kg/day
92	Volume of excess sludge	4.55	m^3/day		9.09	m^3/day		22.74	m^3/day
93	Resulting SRT, θc	13.88	days		13.88	days		13.88	days
94	Least SRT, θc, in design	8.00	days		8.00	days		8.00	days
95	Volume of excess sludge for least θc	7.89	m^3/day		15.78	m^3/day		39.45	m^3/day
96	Excess sludge pump set duty as	7.89	m^3/day		15.78	m^3/day		39.46	m^3/day
97	Recirculation ratio	0.80			0.80			0.80	
98	Return sludge pump set duty as MLD	0.80	MLD		1.60	MLD		4.00	MLD

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2.7.3.1. Design Steps of 2 MLD Activated Sludge Process (ASP)

REFERENCES FOLLOWED:

A. Manual on Sewerage and Sewage Treatment Systems, CPHEEO, November 2013.

B. Waste Water Engineering Treatment and Reuse, Metcalf and Eddy, 4th edition, Tata McGraw Hill edition.

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Input Data and Sewage Characteristics

1. Influent flow rate = Q_o = 2 MLD

   The design is being undertaken for a 2 MLD STP based on ASP technology.

2. Influent flow rate = 2000 m^3 /day

   2 MLD = 2 x 10^6 / 10^3 = 2000 m^3 /day

3. Influent flow rate = 41.6667 m^3 /hour

   2 MLD = 2000 / (24) = 83.3333 m^3 /hour

4. Influent flow rate = 2000 / (24 x 60 x 60) = 0.023148 m^3 /s
5. Elevation of the STP location = 310 m

   The STP is designed for a town in central India with a latitude of 21.14^o N. The average elevation of the town is 310 m as per topographic map.

6. Temperature = 14.5^{o}C

   In the present study, the plant is assumed to be located at a latitude of 21.14^o N (Central India) where the lowest winter temperature is obtained as 14.5^o C from the IMD records.

7. Influent BOD_5 = 87.5 mg/l

   As per the UASBR design, the BOD_5 of the effluent from the UASB reactor is 87.5 mg/l. Since the ASP reactor is in sequence with the UASB reactor, the influent BOD_5 into the ASP reactor shall be 87.5 mg/l

Effluent Standards

8. BOD_5 = 30 mg/l

9. COD = 250 mg/l

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10. TSS = 100 mg/l

With reference to Table 5.3 of the CPHEEO manual, the permitted level of BOD_5, COD, and TSS for discharge into inland surface water bodies is considered. The table is based on General standards for Discharge of Environmental Pollutants, Part A: Effluents. The present design is executed in order to meet the above-mentioned limits.

Design Parameters and Assumptions

11. Sludge thickener overflow return into the reactor = 15%

    Value assigned as per the input data in Appendix A 5-12 of CPHEEO Manual

12. Volume of thickener overflow return into the reactor

    = Percentage return x average flow = 0.15 x 2.0 = 0.3 MLD

13. BOD of the thickener overflow return into the reactor = 500 mg/l

    Value assigned as per the input data in Appendix A 5-12 of CPHEEO Manual

14. Centrate from sludge dewatering as fraction of plant flow = 0.6%

    Value assigned as per the input data in Appendix A 5-12 of CPHEEO Manual

15. Volume of centrate from sludge dewatering

    = centrate as fraction of plant flow x average flow = 0.006 x 2 =0.012 MLD

16. BOD of the centrate from sludge dewatering = 380 mg/l

    Value assigned as per the input data in Appendix A 5-12 of CPHEEO Manual

17. Influent BOD into the aeration tank

    This is the weighted BOD of the wastewater in the reactor, thickener overflow into the reactor and the centrate from sludge dewatering.

    Influent BOD into the aeration tank = $\frac{(2.0\ x\ 87.5)+(0.30 x 500)+(0.012 x 380)}{(2.0+0.30+0.012)}$

    = 142.5433 mg/l

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18. Weighted BOD to be removed from the aeration tank = Influent BOD – Effluent BOD = 142.5433 – 30 = 112.5433 mg/l

19. MLSS = 3000 mg/l (Range is 3000 to 4000 mg/l)

20. F : M Ratio = 0.35 (Range is 0.3 to 0.6)

21. Total Volatile Fraction (MLVSS) of MLSS = 0.8

The MLSS, F:M ratio and the total volatile fraction is taken from the values furnished in Table 5.9 of CPHEEO manual, which is presented below.

Table 5.9 from CPHEEO manual listing the design parameters for the three categories of activated sludge processes is presented below.

Table 5.9 from CPHEEO Manual

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

Determination of Reactor Volume

The aeration tank volume is calculated from three different criteria and the maximum of the three values is adopted.

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i. F : M ratio – 0.30 to 0.60 Kg BOD /day / Kg MLVSS
Adopted value = 0.35

ii. SRT – 5 to 8 days
Adopted value = 8 days

iii. Hydraulic retention time – 4 to 6 hours
Adopted value = 5 hours

Based on F:M ratio

22. Compute the F component of food to microorganism ratio (F:M)
F = Weighted BOD x (average flow + thickener overflow + centrate flow)
= 123.3564 x (2 + 0.3 + 0.012) = 285.2 Kg/day

23. Compute the M component of food to microorganism ratio (F:M)
M = F/(M/F ratio) = 285.2 / 0.35 = 814.8571 Kg

24. Aeration tank volume = M x MLVSS fraction of MLSS / MLSS
= 814.8571 / (0.8 x 3000) x 1000 = 339.5238 m^3

Based on Solids Retention Time (SRT)

25. Mean cell residence time or SRT
θ_c = 8 days
The SRT value is obtained from Figure 5.38 of CPHEEO manual for a temperature of 14.5^{o}C.

26. Constant Y = 0.5
The value of Y is taken from clause 5.8.1.3 of CPHEEO manual.

27. Constant K_d = 0.06 1/day
The value of K_d is taken from clause 5.8.1.3 of CPHEEO manual.

28. Required aeration tank volume based on SRT = 222.2638 m^3.
The calculation of the aeration tank volume is carried out as follows:

$$\theta_C = \frac{VX}{Q_W X_S}\ Equation\ 5.24\ of\ CPHEEO\ manual$$

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$$Q_W = YQ(S_0 - S) - K_d V \quad Equation\ 5.25\ of\ CPHEEO\ manual$$

Insert equation 5.25 into equation 5.24 and simplify to obtain the expression for V as:

$$V = \frac{YQ(S_0 - S)\theta_C}{X(1 + K_d\theta_C)}$$

$$V = \frac{0.5\ (2)(153.3654 - 30)8}{3000(1 + 0.06(8))} = 222.2638\ m^3$$

Based on HRT

29. Hydraulic Retention time (HRT) = 5.0 hours
The HRT time corresponds to the value prescribed in Table 5.9 of CPHEEO manual.

30. Aeration tank volume based on HRT = Q x 1000 x HRT/24
= 2 x 1000 x 5 / 24 = 416.6667 m^3

31. Adopted volume = maximum volume among F:M ratio, SRT and HRT
= 416.6667 m^3/s.

32. From the above calculations it is observed that the required capacity of the aeration tank is 416.6667 m^3. This produces a volumetric BOD loading rate of 0.5921 Kg BOD / day / m^3 which is within the prescribed limit as per Table 8-16 of Metcalf and Eddy.

$$Volumetric\ loading = \frac{BOD_{IN}(Average\ Design\ Flow)}{Volume\ of\ the\ tank} = \frac{123.3654(2000)}{(416.6667)(1000)} = 0.5921$$

This lies between the recommended value of 0.3 – 1.6 Kg BOD / day / m^3

Sizing of the Reactor

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33. Depth of liquid in the reactor tank = 5.5 m (ASSUMED)

As per clause 5.8.1.7.5.1 in CPHEEO manual, 'The depth usually ranges from 3 m to 4.5 m for surface aerators. 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. In the present design the value of 5.5 m from Appendix A – 5.12 is adopted.

34. L/B Ratio = 1.0

As per clause 5.8.1.7.5.1 in CPHEEO manual, 'The width 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'. In light of this, he adopted L/B ratio is 1.0 is satisfactory.

35. Length of the tank = $\sqrt[2]{416.6667/5.5 =}$ 8.7039 m.

36. Width of the tank = 1.0 x 8.6969 = 8.7039 m

37. Adopted length of the tank = 8.75 m

38. Adopted width of the tank = 8.75 m

Determination of Aeration Requirement

39. BOD removed in the aeration tank = (Average flow + Thickener overflow + Centage from sludge dewatering) x weighted BOD

= (2.0 + 0.30 + 0.012) x 123.3546 = 285.2000 Kg/day

40. Oxygen requirement in Kg oxygen / Kg BOD removed.

This method employs rule of thumb in order to obtain the oxygen requirement. As per Table 5.9 of CPHEEO manual, the oxygen requirement is 0.8 to 1.0 Kg O_2/Kg BOD removed. In the present study the oxygen requirement is taken as 0.9 Kg O_2/Kg BOD removed.

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41. Kg of oxygen needed per day = BOD removed per day x Oxygen required per Kg BOD = 285.200 x 0.9 = 256.6800 Kg O_2/day

'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.' Designers adopt 2 mg/l for both conventional and extended aeration plants. The same has been adopted in Appendix A-5.12 of CPHEEO manual.

Required Capacity of Surface Aerators

The oxygen transfer capacity of aerators under field conditions can be calculated from the standard oxygen transfer capacity at 20° C and 760 mm Hg barometric pressure by using equation 5.30 of CPHEEO manual.

$$N = \frac{N_s (C_s - C_L) 1.024^{T-20} \propto}{9.17} \; Equation\ 5.30\ of\ CPHEEO\ manual$$

Where

N: Oxygen transferred under field conditions, Kg O_2 / KWh

N_s: Oxygen transfer capacity under saturated 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/l

T: Operating temperature, ^{o} C

α: Correction factor for oxygen transfer in sewage

42. Correction factor for oxygen transfer for sewage, α = 0.83.
α represents the ratio of the oxygen uptake rate, of the given sewage to that of clean tap water at 20^o C. As per clause 5.8.1.7.5.3 of CPHEEO manual, the value of α has to be chosen judiciously as it impacts the cost of the aeration system. The CPHEEO manual specifies a value of 0.8 to 0.85. Hence the chosen value of 0.83 is OK.

43. Multiplying factor for DO saturation for sewage at operating temperature, β = 0.95.

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As per clause 5.8.1.7.5.3 of CPHEEO manual, the value of β is 0.95 for domestic sewage. Used for calculating C_s.

44. Dissolved oxygen at operating temperature = 10.3673 mg/l

Let operating temperature = T. As per Appendix A – 5.12 of CPHEEO manual,

D.O. at T = $14.42 + 0.003 \times T^2 - 0.323 \times T = 14.42 + 0.003 \times 14.5^2 - 0.323 \times 14.5$

= 10.3673 mg/l

The DO at 14.5° C can also be obtained from Table 5.10 of CPHEEO manual.

45. DO at operating temperature and elevation = 10.0078 mg/l

Let site elevation = Z. As per Appendix A – 5.12 of CPHEEO manual,

DO at operating elevation = (1-(Z / 152) x 0.017) x DO at operating temperature

= (1 – (310 / 152) x 0.017) x 10.3673 = 10.0078 mg/l.

The DO at 310 m elevation can also be obtained from Table 5.11 of CPHEEO manual.

46. Oxygen Tension = ($C_s - C_L$) = 7.5074

As per Appendix A-5.12 of CPHEEO manual the oxygen tension is given by:

$C_s \times \beta$ – Residual oxygen in aerator.

Oxygen Tension = ($C_s - C_L$) = 10.0078 x 0.95 – 2.0 = 7.5074 mg/l

47. Oxygen gradient = 0.8187 mg/l

As per Appendix A-5.12 of CPHEEO manual, Oxygen gradient = ($C_s - C_L$)/9.17

= 7.5074 / 9.17 = 0.8187 mg/l

48. Temperature difference = operating temperature – standard temperature

T – 20 = 14.5-20 = -5.5°C

49. Temperature coefficient = 1.024

This is as per equation 5.30.

50. Temperature correction factor

= $1.024^{(T-20)} = 1.024^{-5.5} = 0.8777$

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51. Conversion factor to standard conditions

$$= \frac{(C_s - C_L)1.024^{T-20} \propto}{9.17}\ Equation\ 5.30\ of\ CPHEEO\ manual$$

= 0.8187 x 0.8777 x 0.83 = 0.5964

52. Oxygen needed under standard conditions = Kg of oxygen needed per day / conversion factor = 256.68 / 0.5964 = 430.3705 Kg/day

53. Factor of safety for aeration intangibles = 1.1

The 10% increase is in accordance to Appendix A – 5.12 of CPHEEO manual

54. Oxygen requirement after incorporating factor of safety as per Appendix – A 5.12 = Oxygen needed under normal conditions x Factor of safety

= 430.3705 x 1.1 = 473.4076 Kg/day

55. Oxygen transfer capacity of aerator 1.8 Kg/KWh

As per clause 5.8.1.7.5.3 of CPHEEO manual, the oxygen transfer capacity of aerator varies between 1.2 to 2.4 Kg/KWh. The average value is taken in the present study.

56. Required aerator capacity = 473.4076 / (1.8 x 24) = 10.9585 KW

Calculation of capacity of Diffused Aerators

The calculation is done as per clause 5.8.1.7.5.4 of CPHEEO manual, by further adjusting the air volume calculated based on the α value for surface aerators. This is necessary as the diffusers are located in the tank at 0.3 to 0.6 m above the tank floor and hence are subjected to a pressure of the overlying water column.

57. Standard Temperature = 20^{o} C

The standard temperature is used to compute the standard oxygen transfer rate (SOTR) from atmosphere to the wastewater.

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58. Density of air at operating temperature is computed using an empirical equation given in Appendix A – 5.12 of CPHEEO manual.

$\rho_{T^oC} = 1.285 + T^3/10^6 - T^2/(10^5 \times 7) - 0.003 \times T$

$\rho_{14.5^oC} = 1.285 + 14.5^3/10^6 - 14.5^2/(10^5 \times 7) - 0.003 \times 14.5 = 1.2442$ Kg/m^3

59. Density of air at 20^o C temperature is computed using an empirical equation given in Appendix A – 5.12 of CPHEEO manual.

$\rho_{T^oC} = 1.285 + T^3/10^6 - T^2/(10^5 \times 7) - 0.003 \times T$

$\rho_{20^oC} = 1.285 + 20^3/10^6 - 20^2/(10^5 \times 7) - 0.003 \times 20 = 1.2324$ Kg/m^3

60. Content of oxygen in the air based on weight = 23. 20% = 0.2320

61. Kg of oxygen needed for residual DO of 2 mg/l per day = 2 x 2 = 4 Kg

62. Total oxygen needed per day = 256.68 + 4.0 = 260.68 Kg/day

63. Air needed = 260.68 / (0.2320 x 1.2442) = 903.0518 m^3/day

64. Oxygen transfer efficiency of diffuser system per meter depth = 0.05

The oxygen transfer efficiency is specified by the manufacturer of the diffuser system and it is taken as 0.05 Per m as per Appendix A – 5.12 of CPHEEO manual.

65. Transfer efficiency at design depth of 5.5 m = 5.5 x 0.05 = 0.275

66. Diffuser fouling factor = 0.04

As per clause 5.8.1.7.5.4 of CPHEEO manual, 'Fouling factor of diffusers at the rate of 4 % to 5 % per year over its life span.'

67. Diffuser life cycle = 3 year

Adopted based on field observations and the specified value as per clause 5.8.1.7.5.4 of CPHEEO manual

68. Diffuser fouling factor, F, over its life cycle = $(1+0.04)^3 = 1.1249$

The life cycle cost is obtained by using standard formula used in literature.

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69. Factor of safety for intangible = FS = 1.1
Taken as per clause 5.8.1.7.5.4 of CPHEEO manual.

70. Air needed for oxygenation in m^3/day = air needed x F x FS / Transfer efficiency at design depth = 903.0518 x 1.1249 x 1.1 / 0.2750 = 4063.2420 m^3/day

71. Air needed per hour = 4063.2420 / 24 = 169.3018 m^3/hour

72. Air mixing criteria = 16 m^3 / minute / 1000 m^3 of tank volume
As per clause 5.8.1.7.5.6 in CPHEEO manual mentioned in Appendix A – 5.12

73. Air required for mixing as per criteria = Volume of aeration tank x 16 x 60 / 1000
= 416.667 x 16 * 60 / 1000 = 400 m^3/hour

74. Minimum air required volume required for mixing = 2.7 m^3/hour/m^2 of floor area
Extracted from EPA 625/8-85/0100, p 38 and mentioned in clause 5.8.1.7.5.6 of CPHEEO manual.

75. Surface area of the aeration tank = Volume / depth = 416.6667 / 5.5 = 75.7576 m^2

76. Volume of air needed as per US EPA guidelines = 75.7576 x 2.7 = 204.5455 m^3/hour

77. Adopted value of air needed for mixing = Maximum of 400, 204.5455 and 169.3018
= 400 m^3/hour

78. Requirement of air under standard conditions = Adopted requirement / Conversion factor = 400 / 0.5964 = 670.6725 m^3/hour

79. Accounting for friction and other losses as fraction of depth = 0.2

80. Liquid depth for design of compressor capacity = depth x (1 + friction loss factor)
= 5.5 x (1 + 0.2) = 6.6 m

81. Required capacity of the compressor at 1400 rpm

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= 0.746 x ((0.03 x air volume/hour) + 16))

= 0.746 x ((0.03 x 670.6725) + 16) = 26.9457 KW

As per Appendix A 5.12, for DPR purpose equation for compressor KW at 1400 rmp can be taken as:

i. For 7 m water column, BHP = 0.03*(cum / hr)+16

ii. For 6 m water column, BHP = 0.025*(cum / hr)+13

iii. For 5 m water column, BHP = 0.02*(cum / hr)+14

Sludge Production Calculations

82. As per clause 5.8.1.7.5.10 of CPHEEO manual, the excess sludge is obtained using the equation:

$$A = Q\,Y_{obs}(S_o - S)$$

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

Y = 0.5

K_d = 0.06

$$Y_{obs} = \frac{Y}{(1 + K_d\theta_c)} = \frac{0.5}{(1 + 0.06x8)} = 0.3378$$

83. Excess sludge mass wasted = Y_{obs} x weighted BOD x average flow

= 0.3378 x 123.3564 x 2 = 83.3489 Kg/day

84. Ratio of excess sludge to BOD removed = 0.4

As per clause 5.8.1.7.5.10 of CPHEEO manual, ‘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. Hence the adopted value of 0.4 is OK.

85. Excess sludge from thumb rule = Q x Weighted BOD x Ratio of excess sludge to BOD

= 2 x 123.3564 x 0.4 = 98.6851 Kg/day

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86. Adopted value of excess sludge = Higher of (83) and (85) = 98.6851 Kg/day

87. Concentration factor for MLSS in return / excess sludge = 3.3
As per value furnished in Appendix Appendix A 5.12.

88. Return / excess sludge MLSS concentration = concentration factor x MLSS
= 3.3 x 3000 = 9900 mg/l

89. Cells in aeration tank = MLSS x Volume of aeration tank / 1000
= 3000 x 416.6667 / 1000 = 1250 Kg

90. Cells wasted per day from the system = Excess sludge = 98.6851 Kg/day

91. Volume of Excess sludge = 98.6851 x 10^6/(9900 x 1000) = 9.9682

92. Resulting SRT = θ_c = Cells in aeration tank / Cells wasted per day
= 1250 / 96.6851 = 12.6665 days.

93. Least SRT (Design, Resulting) = Least (8, 12.6665) = 8 days.

94. Volume of excess sludge for least SRT = 9.9682 x 12.6665 / 8 = 15.7828 m^3/day

95. Excess sludge pump set duty = 15.7828 m^3/day

96. Recirculation ratio = 0.8
As per Table 5.9 of CPHEEO manual, recirculation ratio varies between 0.25 & 0.8.
The maximum value is considered in design.

97. Return sludge pump set duty as MLD = 0.8 x 2 = 1.6 MLD.

Table 2.23 Summary of the design of UASB + post treatment through ASP

Component	Capacity		
	1 MLD`	2 MLD	5 MLD
UASB Reactor Details			
No. of UASB reactors	1	1	1
L (m)	11.5	15.9	25.0

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B (m)	5.6	8.2	12.5
D (m)	7.0	7.0	7.0
ASP Reactor Details			
No. of aerator tanks	1	1	1
L (m)	6.25	8.75	14.0
B (m)	6.25	8.75	14.0
D (m) + 0.5 m freeboard	6.0	6.0	6.0
Aeration required			
m^3/hour	335.3363	670.6725	1676.6813

2.8 Design of Sequencing Batch Reactor (SBR)

Sequencing batch reactors (SBRs), also known as sequential batch reactors, are activated sludge batch reactors, unlike most of the other technologies which have plug flow reactors. SBRs has proved to be very popular with many ULBs in the country owing to their low footprint on account of integrating separate functions into a single treatment tank.

2.8.1 Working of SBR

Standard SBR setups usually include the reactor basin, sludge draw-off mechanism, effluent decanter, and process control system. As a batch reactor, SBR is a closed system where no flow is added or allowed to go out during the detention period. Reaction kinetics of a batch reactor are of first order and are based on the assumption that there is a complete mixing of reactants and their concentration is uniform throughout the reactor. There is considerable flexibility in fixing the timings of aeration, settling, decanting and idle periods. In many cases, the idle phase is eliminated without any adverse effect on the process.

SBRs process wastewater in batches through single fill-and-draw tanks. Some designs may use two or more batch tanks in optimizing the effectiveness of their systems, depending on the expected volume of wastewater flow and treatment period.

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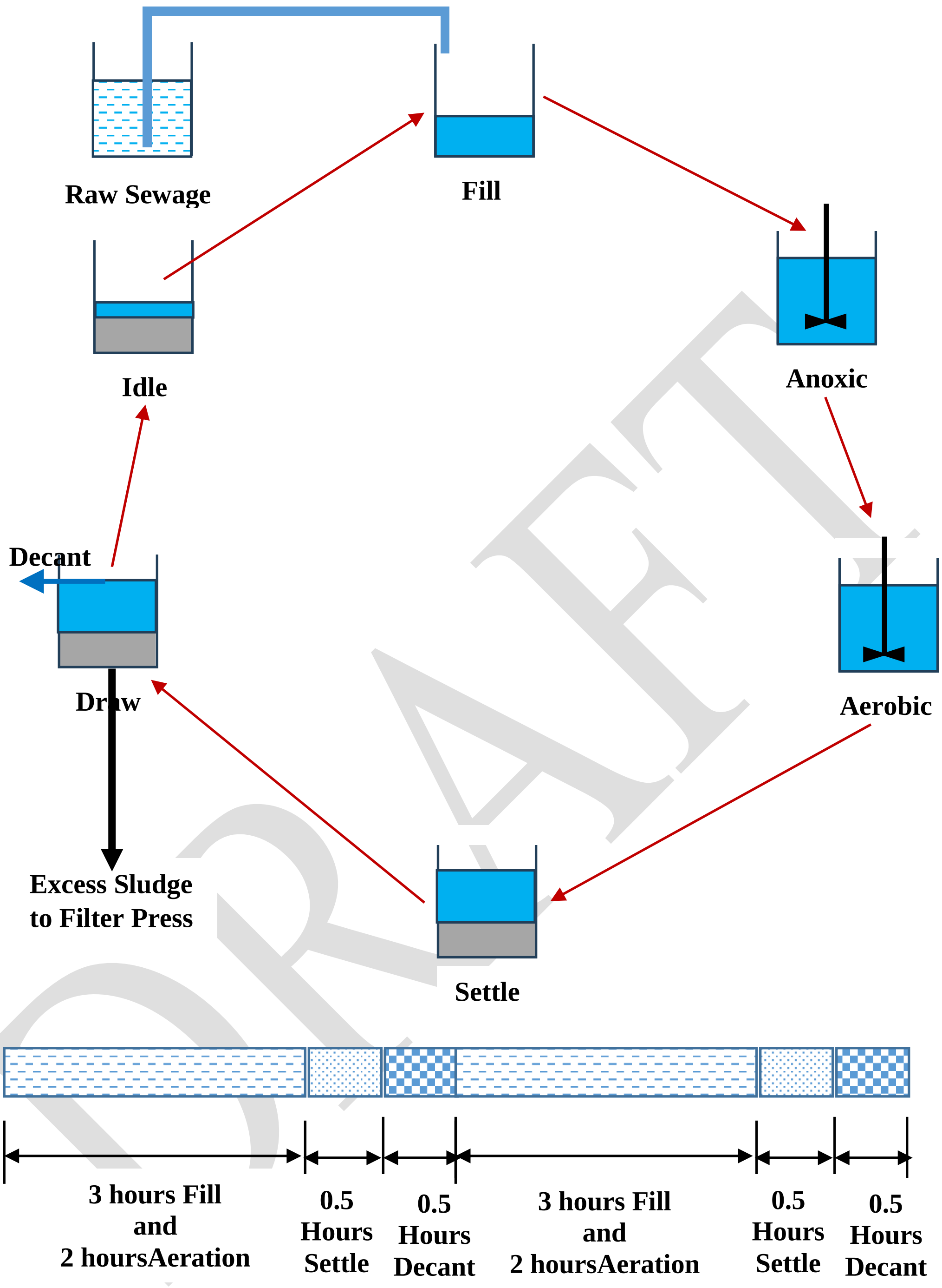

Figure 2.13 Flowchart showing the concept of timing of sequence of operations

Each tank operates under five commands of fill, react, settle, draw, and idle. Screening and grit removal is undertaken prior to introducing the wastewater in to the SBR reactor. In a new plant, each batch reactor is partially filled with biomass to facilitate the aerobic processes. Figure 2.13 shows the sequential processes of an SBR. The idle phase may sometimes be done away

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with and instead aeration may be continued during half of the fill phase duration also. Typical values of the different stages are also shown at the bottom of the figure.

2.8.2 Design calculations of SBR

The design calculations of 2 MLD SBR reactor are presented in Table 2.24. The design is based on 'Wastewater Engineering, Treatment and Reuse' by Metcalf and Eddy, 4th edition, 2003. The design parameters are also taken from the same reference book. Relevant requirements of CPHEEO manual are also adopted. The summary of the design outcome for 1 MLD, 2 MLD and 5 MLD SBR plants is presented in Table 2.26.

Table 2. 22 Design calculations of SBR Reactor

DESIGN OF SBR REACTOR OF CAPACITY 1, 2 AND 5 MLD									
S. No.	**Item**	**1 MLD**			**2 MLD**			**5 MLD**	
		Value	**Unit**		**Value**	**Unit**		**Value**	**Unit**
	INPUT DATA AND SEWAGE CHARACTERISTICS								
1	Inflow rate into the STP	1.00	MLD		2.00	MLD		5.00	MLD
2	Inflow rate into the STP	1000.00	m^3/day		2000.00	m^3/day		5000.00	m^3/day
3	Inflow rate into the STP	0.01157	m^3/s		0.02315	m^3/s		0.05787	m^3/s
4	Peak factor	3.00	Unit less		3.00	Unit less		3.00	Unit less
5	Peak flow into the STP	3.00	MLD		6.00	MLD		15.00	MLD
6	Peak flow into the STP	3000.00	m^3/day		6000.00	m^3/day		15000.00	m^3/day
7	Peak flow into the STP	0.035	m^3/s		0.069	m^3/s		0.176	m^3/s
8	Temperature	14.50	°C		14.50	°C		14.50	°C
9	X, Reactor mixed liquor concentration (MLSS)	3500.00	mg/l		3500.00	mg/l		3500.00	mg/l

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10	Influent BOD	250.00	mg/l		250.00	mg/l		250.00	mg/l
11	sBOD	90.00	mg/l		90.00	mg/l		90.00	mg/l
12	COD	425.00	mg/l		425.00	mg/l		425.00	mg/l
13	sCOD	115.00	mg/l		115.00	mg/l		115.00	mg/l
14	rbCOD	70.00	mg/l		70.00	mg/l		70.00	mg/l
15	TSS	375.00	mg/l		375.00	mg/l		375.00	mg/l
16	VSS	262.50	mg/l		262.50	mg/l		262.50	mg/l
	DESIGN PARAMTERS AND ASSUMPTIONS								
17	Number of tanks to be used	2.00	No.		2.00	No.		2.00	No.
18	Total liquid depth when full	6.00	m		6.00	m		6.00	m
19	Decant depth percentage	30.00	%		30.00	%		30.00	%
20	SVI	150.00	mg/l		150.00	mg/l		150.00	mg/l
21	Minimum D.O. concentration in Reactor	2.00	mg/l		2.00	mg/l		2.00	mg/l
22	bCOD/BOD Ratio	1.60			1.60			1.600	
	EFFLUENT STANDARDS								
23	Effluent TSS	100.00	mg/l		100.00	mg/l		100.00	mg/l
24	Effluent BOD	30.00	mg/l		30.00	mg/l		30.00	mg/l
25	Effluent COD	250	mg/l		250	mg/l		250	mg/l
26	Estimated Effluent COD	73.00	mg/l		73.00	mg/l		73.00	mg/l
	STANDARD KINETIC COEFFICIENTS								

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27	μ_m	6.00	g VSS/g VSS.d		6.00	g VSS/g VSS.d		6.00	g VSS/g VSS.d
28	K_s	20.00	g bCOD/m^3		20.00	g bCOD/m^3		20.00	g bCOD/m^3
29	Y	0.40	g VSS/g bCOD		0.40	g VSS/g bCOD		0.40	g VSS/g bCOD
30	k_d	0.12	g VSS/g VSS.d		0.12	g VSS/g VSS.d		0.12	g VSS/g VSS.d
31	f_d	0.15	unit less		0.15	unit less		0.15	unit less
	θ VALUES								
32	μ_m	1.070	unit less		1.07	unit less		1.07	unit less
33	K_s	1.00	unit less		1.00	unit less		1.00	unit less
34	k_d	1.04	unit less		1.04	unit less		1.04	unit less
	WASTEWATER DESIGN PARAMETERS								
35	bCOD	400.00	mg/l		400.00	mg/l		400.00	mg/l
36	bpCOD / pCOD	0.82	unit less		0.82	unit less		0.82	unit less
37	nbVSS	45.72	mg/l		45.72	mg/l		45.72	mg/l
38	iTSS	112.50	mg/l		112.50	mg/l		112.50	mg/l
	CHOICE OF SBR OPERATING CYCLE								
39	Reaction or Aeration time t_A	2.00	hours		2.00	hours		2.00	hours
40	Settling time t_S	0.50	hours		0.50	hours		0.50	hours
41	Decantation time t_D	0.50	hours		0.50	hours		0.50	hours
42	Idle time t_I	0.00	hours		0.00	hours		0.00	hours
43	Fill period t_F for each tank	3.00	hours		3.00	hours		3.00	hours
44	Total cycle time T_c	6.00	hours		6.00	hours		6.00	hours

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45	No. of cycles / tank per day	4.00	cycles / day		4.00	cycles / day		4.00	cycles / day
46	Total No. of cycles / day for 2 tanks	8.00	cycles / day		8.00	cycles / day		8.00	cycles / day
47	Fill volume / cycle	375.00	m^3/fill		750.00	m^3/fill		1875.00	m^3/fill
	DETERMINATION OF FILL FRACTION								
	Mass Balance equation $V_TX = V_SX_S$								
	V_T = Total Volume m^3								
	X = MLSS concentration at full volume, mg/l								
	V_S = Settled volume after decantation, m^3								
	X_S = MLSS concentration in settled volume mg/l								
48	X_S based on the SVI value	6666.67	mg/l		6666.67	mg/l		6666.67	mg/l
49	$V_S / V_T = X / X_S$	0.52	unit less		0.52	unit less		0.52	unit less
50	Percent liquid above sludge blanket to avoid removal of solids during decantation	20.00	%		20.00	%		20.00	%
51	V_S / V_T	0.63	unit less		0.63	unit less		0.63	unit less
	V_F = Fill volume								
	$V_T = V_F + V_S$ Total Volume = Fill Volume + Settled Volume after Decantation								
52	V_F / V_T	0.37	unit less		0.37	unit less		0.37	unit less
53	Compare V_F / V_T with Decant Depth %	OK			OK			OK	
54	Full liquid depth	6.00	m		6.00	m		6.00	m

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55	Decant depth	1.80	m		1.80	m		1.80	m
56	V_T	1250.00	m^3/tank		2500.00	m^3/tank		6250.00	m^3/tank
57	Overall time τ	20.00	hours		20.00	hours		20.00	hours
	DETERMINE THE TANK DIMENSIONS								
58	Tank freeboard	0.30	m		0.30	m		0.30	m
59	Total tank depth	6.30	m		6.30	m		6.30	m
60	Length to width ratio of the tank	1.00	unit less		1.00	unit less		1.00	unit less
61	Length of the tank	14.43	m		20.41	m		32.27	m
62	Width of the tank	14.43	m		20.41	m		32.27	m
63	Adopted Length of the tank	14.50	m		20.50	m		32.50	m
64	Adopted width of the tank	14.50	m		20.50	m		32.50	m
65	Adopted Volume of one tank	1261.50	m^3		2521.50	m^3		6337.50	m^3
	DETERMINATION OF SRT USING TRIAL AND ERROR SOLUTION								
	Determine SRT using Equations from Metcalf and Eddy								
66	SRT (Obtained by solving Equation 8.16)	11.06	What if Analysis Outcome		11.06	What if Analysis Outcome		11.06	What if Analysis Outcome
67	k_d @ Given Temperature	0.09672	1/day		0.09672	1/day		0.09672	1/day
68	$(P_{X,TSS})SRT = V \times X_{MLSS}$	4375000.00	g		8750000.00	g		21875000.00	g
69	Assume $S_o = S_o - S$								
70	A/0.85	1508666.90			3017333.80			7543334.49	
71	B/0.85	242008.94			484017.87			1210044.68	

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72	D	758405.61		1516811.21		3792028.04	
73	E (Inert TSS in influent)	1865918.58		3731837.11		9329592.79	
74	Statement of Equation 8.16 to be reduced to '0'	-0.00002	Cell to be set equal to '0' in What if Analysis	-0.00004	Cell to be set equal to '0' in What if Analysis	-0.00010	Cell to be set equal to '0' in What if Analysis
	DETERMINE MLVSS CONCENTRATION						
75	A	1282366.86		2564733.73		6411834.32	
76	B	205707.60		411415.19		1028537.98	
77	D	758405.61		1516811.21		3792028.04	
78	A+B+C+D = $V_T X_{MLVSS}$	2246480.07	g/day	4492960.13	g/day	11232400.34	g/day
79	X_{MLVSS} (MLVSS Concentration)	1797.18	mg/l	1797.18	mg/l	1797.18	mg/l
80	Fraction of MLVSS (X_{MLVSS}/X_{MLSS})	0.51	unit less	0.51	unit less	0.51	unit less
	DETERMINE BIOMASS AS VSS WASTED PER DAY						
81	A	115974.73		231949.47		579873.67	
82	B	18603.79		37207.58		93018.95	
83	$P_{X,BIO}$ = A + B	134578.52	g/day	269157.05	g/day	672892.62	g/day
84	$P_{X,BIO}$ = A + B	134.58	Kg/day	269.16	Kg/day	672.89	Kg/day
	DETERMINE THE DECANT PUMPING RATE						
85	Pumping rate	12.50	m^3/minute	25.00	m^3/minute	62.50	m^3/minute
	DETERMINE OXYGEN REQUIRED PER TANK						

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86	R_o	408.90	kg/day/tank	817.80	kg/day/tank	2044.49	kg/day/tank
87	Total Aeration time per tank	12.00	hours/day	12.00	hours/day	12.00	hours/day
88	Average oxygen transfer rate	34.07	Kg/hour	68.15	Kg/hour	170.37	Kg/hour
89	Factor for higher oxygen in the beginning	1.50	unit less	1.50	unit less	1.50	unit less
90	Practical average oxygen transfer rate	51.11	Kg/hour	102.22	Kg/hour	255.56	Kg/hour
	DETERMINE SLUDGE PRODUCTION						
91	$P_{X,TSS}$ Sludge production rate	791.33	Kg/day	1582.66	Kg/day	3956.66	Kg/day
92	bCOD removal rate	1200.00	Kg/day	2400.00	Kg/day	6000.00	Kg/day
93	BOD Removal rate	750.00	Kg/day	1500.00	Kg/day	3750.00	Kg/day
94	TSS Observed yield	1.06	g TSS/g BOD	1.06	g TSS/g BOD	1.06	g TSS/g BOD
95	VSS Observed yield	0.54	g VSS/g TSS	0.54	g VSS/g TSS	0.54	g VSS/g TSS
96	TSS / bCOD Observed yield	0.66	g TSS / g bCOD	0.66	g TSS / g bCOD	0.66	g TSS / g bCOD
97	Sludge wasting rate from full tank at MLSS concentration	226.10	m^3/day	452.20	m^3/day	1130.48	m^3/day
98	Sludge wasting rate from settled sludge	118.70	m^3/day	237.40	m^3/day	593.50	m^3/day
	DETERMINE F/M AND BOD VOLUMETRIC LOADING						
99	F/M	0.17	g/g-day	0.17	g/g-day	0.17	g/g-day
100	BOD Volumetric Loading	0.30	Kg/m^3.day	0.30	Kg/m^3.day	0.30	Kg/m^3.day
	DETERMINE AIR FLOW RATE AS PER RULE OF THUMB						
101	Oxygen Requirement a per rule of thumb	30.25	Kg/hour	60.50	Kg/hour	151.25	Kg/hour

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102	Standard oxygen transfer rate for diffusers (SOTR)	35.42	%		35.42	%		35.42	%
103	Actual oxygen transfer rate under field conditions (AOTR)	11.70	%		11.70	%		11.70	%
104	Required average oxygen transfer rate during aeration AOTR	45.38	Kg/hour/tank		90.75	Kg/hour/tank		226.88	Kg/hour/tank
105	Air Requirement	1324.76	Nm^3/hour		2649.52	Nm^3/hour		6623.80	Nm^3/hour
106	Factor for increase of oxygen transfer rate at the beginning of the cycle	1.50	unit less		1.5	unit less		1.50000	unit less
107	Design airflow rate	1987.14	Nm^3/hour		3974.28	Nm^3/hour		9935.70	Nm^3/hour
108	Blower Outlet Pressure	1.60	bar		1.60	bar		1.60	bar
	DETERMINE AIR FLOW RATE AS PER BIOLOGICAL KINETICS USING DIFFUSERS								
109	Average oxygen transfer rate per tank	17.04	Kg/hour/tank		34.07	Kg/hour/tank		85.19	Kg/hour/tank
110	Required average oxygen transfer rate during aeration AOTR	51.11	Kg/hour/tank		102.22	Kg/hour/tank		255.56	Kg/hour/tank
	Bubble Aeration Design								
111	P_b / P_a computed as per equation in Appendix B of Metcalf and Eddy	0.96	unit less		0.96	unit less		0.96	unit less
112	Oxygen concentration at design temperature and altitude, $C_{S,T,H}$	9.81	mg/l		9.81	mg/l		9.81	mg/l

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113	Atmospheric pressure in m of water at design temperature and altitude as per Appendix B and C of Metcalf and Eddy	9.97	m of water		9.97	m of water		9.97	m of water
114	Average DO saturation concentration in clean water in aeration tank at design $C_{\bar{S},T,H}$ temperature and altitude	12.00	mg/l		12.00	mg/l		12.00	mg/l
115	Determine SOTR	89.26	Kg/hour		178.53	Kg/hour		446.32	Kg/hour
116	Density of air at design temperature	1.18	Kg/m^3		1.18	Kg/m^3		1.18	Kg/m^3
117	Amount of oxygen by weight	0.27	Kg/m3		0.27	Kg/m3		0.27	Kg/m3
118	Determine the airflow rate	15.50	m^3/minute		31.00	m^3/minute		77.50	m^3/minute
119	Air flowrate	930.05	Nm^3/hour		1860.10	Nm^3/hour		4650.24	Nm^3/hour
120	Factor for increase of oxygen transfer rate at the beginning of the cycle	1.50	unit less		1.50	unit less		1.50	unit less
121	Design airflow rate	1395.07	Nm^3/hour		2790.14	Nm^3/hour		6975.36	Nm^3/hour
122	Blower Outlet Pressure	1.60	bar		1.60	bar		1.60	bar
	DIFFUSER DATA								
123	Oxygen needed per Kg BoD	1.10	Kg O_2 / Kg BOD		1.10	Kg O_2 / Kg BOD		1.10	Kg O_2 / Kg BOD
124	SOTR as function of depth	6.56	% per m depth		6.56	% per m depth		6.56000	% per m depth
125	AOTR/SOTR	0.33	unit less		0.33	unit less		0.33	unit less
126	Depth of diffusers	5.40	m		5.40	m		5.40	m
127	Specific weight of water at design temperature from	9.80	KN/m^3		9.80	KN/m^3		9.80	KN/m^3

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	Table C-1 of Metcalf and Eddy								
128	Design Temperature	14.50	ºC		14.50	ºC		14.50	ºC
129	Oxygen content in air	0.29	Kg/m³		0.29	Kg/m³		0.29	Kg/m³
130	Percentage oxygen content in air by weight	0.23	%		0.23	%		0.23	%
131	Diffuser Fouling Factor F	0.90	unit less		0.90	unit less		0.90	unit less
132	Ratio of oxygen transfer rate in wastewater to that in clean water α	0.70	unit less		0.70	unit less		0.70	unit less
133	Ratio of D.O. saturation in wastewater to that in clean water, B	0.95	unit less		0.95	unit less		0.95	unit less
134	Elevation of the town	310.00	m		310.00	m		310.00	m
135	DO saturation concentration in clean water at 20ºC and 1 atm pressure from Table D-1 in appendix D of Metcalf and Eddy	9.08	mg/l		9.08	mg/l		9.08	mg/l
136	D.O concentration at the design temperature as per Table D1 of Appendix D of Metcalf and Eddy (By interpolation)	10.18	mg/l		10.18	mg/l		10.18	mg/l
137	Standard Atmospheric Pressure	101.32	KN/m²		101.32	KN/m²		101.32	KN/m²

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138	Percentage oxygen concentration leaving the tank	19.00	%		19.00	%		19.00	%
139	Efficiency of fine bubble diffuser	35.00	%		35.00	%		35.00	%

	Variable input
	Input from CPHEEO manual or other standards
	Output

2.8.2.1 Design Steps of 2 MLD Sequencing Batch Reactor (SBR)

REFERENCES FOLLOWED

A. Waste Water Engineering Treatment and Reuse, Metcalf and Eddy, 4th edition, Tata McGraw Hill edition.

B. Manual on Sewerage and Sewage Treatment Systems, CPHEEO, November 2013.

Input Data and Sewage Characteristics

1. Influent flow rate = Q_o = 2 MLD

   The design is being undertaken for a 2 MLD STP based on SBR technology.

2. Influent flow rate = 2000 m^3 /day

   2 MLD = 2 x 10^6 / 10^3 = 2000 m^3 /day

3. Influent flow rate = 2000 / (24 x 60 x 60) = 0.023148 m^3 /s

4. Peak factor = 3

5. Peak influent flow rate = 2 x 3 = 6 MLD

6. Peak influent flow rate = 6 x 10^6 / 10^3 = 6000 m^3 /day

7. Peak influent flow rate = 6000 / (24 x 60 x 60) = 0.069444 m^3 /s

8. Temperature = 14.5^{o}C

   In the present study, the plant is assumed to be located at a latitude of 21.14o N (Central India) where the lowest winter temperature is obtained as 14.5 C from the IMD records.

   T = 14.5o C

9. MLSS = 3500 mg/l

   As per Table 5.57 of CPHEEO manual, the MLSS range is 3500 – 5000 mg/l. Hence the chosen value of 3500 mg/l is OK.

10. Influent BOD_5 = 250 mg/l

    As per Table 5.4 of the CPHEEO manual:

    Per capital contribution of BOD_5 = 27.0 g /c / day

    With 135 lpcd water supply and sewage generation at 80% of water supplied, the BOD_5 concentration is obtained as:

    BOD_5 = 27 x 1000 / (135 x 0.8) = 250 mg/l.

11. Soluble BOD = sBOD = 90 mg / l

    The sBOD is a part of BOD_5 and represents that faction of BOD which is due to dissolved organic matter. The remaining BOD is due to particulate matter.

    The sBOD is not specified in the CPHEEO manual. It has to be determined by conducting laboratory test on sewage sample. In the present study, sBOD is taken as 90 mg / l based on the data furnished in example 8-3 of Metcalf and Eddy. This value can be replaced with the actual sBOD as received from laboratory test report.

12. COD of influent = 425 mg/l

    The value of COD of influent sewage is taken from Table 5.4 of CPHEEO manual.

13. Soluble COD = sCOD = 115 mg / l

    The soluble COD is that component of COD which is in dissolved form and hence requires biological treatment for its removal. In contrast, insoluble COD is contributed by particulate matter and can be remove via filtration and / or settlement. The sCOD is not specified in the CPHEEO manual. It has to be determined by conducting laboratory

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test on sewage sample. In the present study, sCOD is taken as 115 mg / l based on the data furnished in example 8-3 of Metcalf and Eddy. This value can be replaced with the actual sCOD as received from laboratory test report.

14. Influent rbCOD = 70 mg/l

The readily biodegradable COD (rbCOD) consists of small molecules that are directly available for biodegradation by heterotrophic microorganisms (volatile fatty acids, alcohols, amino-acids, simple sugars). The rbCOD is not specified in the CPHEEO manual. It has to be determined by conducting laboratory test on sewage sample. In the present study, rbCOD is taken as 90 mg / l based on the data furnished in example 8-3 of Metcalf and Eddy. This value can be replaced with the actual rbCOD as received from laboratory test report.

15. Influent Total Suspended Solids (TSS) = 375 mg/l

The value of TSS of influent sewage is taken from Table 5.4 of CPHEEO manual.

16. Influent Volatile Suspended Solids (VSS) = 262.5 mg/l

The value of VSS of influent sewage is taken from Table 5.4 of CPHEEO manual.

Design Parameters and Assumptions

17. Number of tanks = 2

In order to ensure continuous operation of the plant, a minimum of 2 tanks are required.

18. Total depth of liquid in the tank when full = 6.0 m

19. Decant depth = 30% of tank depth (Assumed)

With reference to Figure 8-16 in Metcalf and Eddy, the decant percentage of maximum value is 35 to 25.

As per clause 5.18.12.1 of CPHEEO manual, the fill-react sequence typically ranges from 30 to 50%.

Hence the assumed value of 30% is OK.

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20. Sludge Volume Index = 150 ml/g

As per clause 5.8.1.7.5.9 of CPHEEO Manual, '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.

Hence the assumed value of 150 ml /g is on the higher side and hence OK.

21. Minimum DO concentration for the aeration basin = 2.0 mg/l

As per Table 8.12 in the Metcalf and Eddy, 'A minimum DO concentration of 2.0 mg/L is recommended for nitrification.'

22. (bCOD) / BOD ratio = 1.6

The biodegradable COD (bCOD) consists of the total fraction from COD that can be biodegraded by the heterotrophic microorganisms. Therefore it is the more proper parameter than total COD for estimating oxygen requirements.

As per equation 8.8 in Metcalf and Eddy, the ratio of bCOD : BOD is 1.6.

Effluent Standards

23. TSS = 100 mg/l

24. BOD = 30 mg/l

25. COD = 250 mg/l

With reference to Table 5.3 of the CPHEEO manual, the permitted level of BOD_5, COD and TSS for discharge into inland surface water bodies is considered. The table is based on General standards for Discharge of Environmental Pollutants, Part A: Effluents. The present design is executed in order to meet the above mentioned limits.

26. The estimated effluent COD is obtained as 73 mg/l which is less than 250 mg/l.

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$$COD_{IN} - \left(\frac{bCOD}{BOD}\right)(BOD_{IN})\left(\frac{BOD_{IN} - BOD_{OUT}}{BOD_{IN}}\right)$$

$$= 425 - (1.6)(250)\left(\frac{250 - 30}{250}\right) = 73\ mg/l$$

Standard Kinetic Coefficients for Activated sludge

27. to 34.

The activated sludge kinetic coefficients for heterotrophic bacteria at 20^{o} C are presented in Table 8.10 of Wastewater Engineering Treatment and Reuse by Metcalf and Eddy, 4th edition. The same is presented below as Table 2.25 for design purpose.

Table 8.10 from Metcalf and Eddy

Table 2.25 Kinetic coefficients for heterotrophic bacteria at 20^{0} C

Coefficient	Unit	Range	Typical value
μ_m	g VSS/g VSS·d	3.0–13.2	6.0
K_s	g bCOD/m^3	5.0–40.0	20.0
Y	g VSS/g bCOD	0.30–0.50	0.40
k_d	g VSS/g VSS·d	0.06–0.20	0.12
f_d	Unitless	0.08–0.20	0.15
θ values			
μ_m	Unitless	1.03–1.08	1.07
k_d	Unitless	1.03–1.08	1.04
K_s	Unitless	1.00	1.00

Determine wastewater characteristics needed in the process design calculations.

35. Determine bCOD using equation 8.8 of Metcalf and Eddy

bCOD = 1.6 BOD = 1.6 x 250 = 400 mg/l

36. Determine Non-biodegradable Volatile Suspended Solids (nbVSS) concentration using equation 8.4 of Metcalf and Eddy,

$$\frac{bpCOD}{pCOD} = \frac{\frac{bCOD}{BOD} x(BOD - sBOD)}{COD - sCOD} \; Equation\ 8.4\ of\ Metcalf\ and\ Eddy$$

Where

bpCOD = concentration of biodegradable particuilate COD, mg/l

pCOD = Concentration of particulate COD, mg/l

sCOD = Concentration of soluble COD in activated sludge effluent, mg/l

$$\frac{bpCOD}{pCOD} = \frac{1.6\ x\ (250 - 90)}{425 - 115} = 0.826$$

37. nbVSS = (1- (bpCOD/pCOD))VSS

= (1 – 0.826) x 262.5 = 45.726 mg/l

38. Determine inert Total Suspended solids using the equation

iTSS = TSS_o – VSS_o = 375 – 262.5 = 112.5 mg/l

Finalize the SBR operating cycle

Let

T_c = Total cycle time

t_F = Fill time, i.e. time taken to fill the tank

t_A = aeration time in the reactor

t_s = Settling time

t_D = Decantation time

t_i = Idle time

Thus $T_c = t_F + t_A + t_S + t_D + t_i$

At least two tanks are needed for continuous operation so that when one tank is in the fill phase t_F the following cycles are occurring in the other tank.

$t_F = t_A + t_S + t_D$

The values of various times are selected based upon prevalent practice as presented in Table 5.57 of CPHEEO's manual and Figure 8-16 of Wastewater Engineering Treatment and Reuse, Metcalf and Eddy.

39. Reaction or aeration time, t_A = 2 hours

40. Settling time, t_s = 0.50 hours

41. Decantation time, t_D = 0.5 hours

42. Idle time, t_i = 0 hours

43. Fill period t_F = 2 + 0.5 + 0.5 = 3 hours for each tank.

44. Total cycle time, $T_c = t_F + t_A + t_S + t_D$ = 3 + 3 = 6 hours

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Table 5.57 from CPHEEO Manual

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%.

45. Number of cycles / tank / day = $\frac{24\ hours/day}{6\ hours/cycle}$ = 4

46. Total number of cycles / day = (2 tanks) (4 cycles / day.tank) = 8 cycles / day

47. Fill volume / cycle = Peak flow / No. of cycles per day = 6000 / 8 = 750 m^3 / fill

Determine the fill fraction per cycle and compare with the selected value of 0.3.

The following calculations are carried out with reference to Figure 2.14. Develop a mass balance equation based on solids in the reactor.

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Mass of solids at full volume = Mass of settled solids

$V_T X = V_S X_S$

Where

V_T = Total volume in m^3

X = MLSS concentration at full volume, mg/litre or g/m^3 = 3500 mg/l

V_S = Settled volume after decant in m^3

X_S = MLSS concentration in settled volume

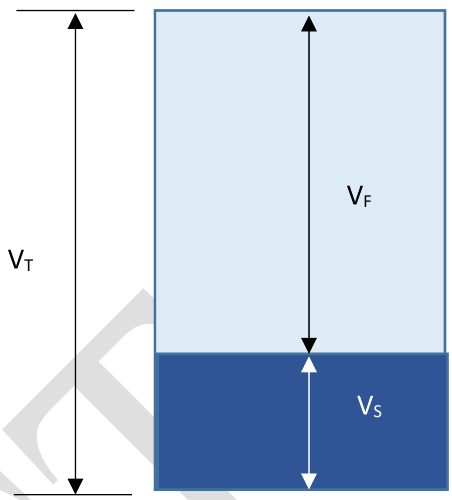

Figure 2.14 Definition of volumes

48. With SVI = 150 ml / g

X_S = 1000 x 1000 / 150 = 6666.67 mg/l or g/m^3

49. $V_s / V_T = X / X_S$ = 3500 / 6666.67 = 0.5250

50. Provide 20% liquid above the sludge blanket so that solids are not removed by the decanting mechanism.

51. V_s / V_T = 1.2 x 0.525 = 0.63

52. Fill fraction is obtained using the equation

$V_F + V_S = V_T$

$$\frac{V_F}{V_T} = 1 - \frac{V_S}{V_T} = 1 - 0.63 = 0.37$$

53. Hence the selected $\frac{V_F}{V_T} = 0.3 < 0.37$ is acceptable.

54. Full liquid depth = 6.0 m

Already presented in (18) above

55. Decant depth = Decant depth percentage x Full liquid depth

= 0.3 x 6.0 = 1.8 m

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56. Total volume per tank is obtained from

V_T = Fill volume per cycle / decant depth percentage

= 750 / 0.3 = 2500 m^3 / tank

57. Overall time, τ is given by the equation

$$\tau = \frac{No. of\ tanks\ x\ Volume\ of\ each\ tank\ x\ (24\ \ hours/day)}{Q}$$

$$\tau = \frac{2\ x\ 2500\ x\ 24}{6000} = 20\ hours$$

Determine Tank Dimensions

58. Free board = 0.3 m (Assumed)

59. Total Tank depth, D = 6.0 + 0.3 = 6.3 m

60. Length to width ratio = 1.0

61. Area required = Volume / depth = 2500 / 6 = 416.67 m^2

Adopt a square tank

Length of the tank = √416.67 = 20.412 m

62. Width of the tank = 20.412 m

63. Adopt length of the tank = 20.5 m

64. Adopted width of the tank = 20.5 m

65. Overall adopted volume of one tank = 20.5 x 20.5 x 6 = 2521.50 m^3 > 2500 m^3

Determine the Solids Retention Time (SRT) Using Trial and Error Solution

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Determination of SRT is done by using equations 7-58, 8-15 and 8-16 of Metcalf and Eddy, 4th edition.

66. SRT = 1.0 days

Assumed a starting value for SRT and solve equation 8.16 by trial and error using the 'Goal Seek' function in Data Tab of MS Excel.

67. $K_{d@14.5^oC}$ temperature = $K_{d@20^oC}$ x $1.04^{(T-20)}$

$= 0.12 \times 1.04^{(14.5-20)} = 0.0967$

68. $(P_{X,TSS})$ SRT = V_T x X_{MLSS}

= 2500 x 3500 = 8750000 g

69. Assume $S_o = S_o - S$

Oxygen used = bCOD removed – COD of waste sludge - Equation 7.58 of M & E

$R_o = Q\,(S_o - S) - 1.42\,P_{X,bio}$ – Equation 7-59 of Metcalf and Eddy

Where

R_o = Oxygen required in Kg/day

$P_{X,bio}$ = biomass as VSS wasted per day, Kg/day

The biomass in the above equation is calculated by using Equation 8-15 of Metcalf and Eddy.

$$P_{X,VSS} = \frac{QY(S_o - S)}{1 + K_d(SRT)} + \frac{f_d k_d YQ(S_o - S)(SRT)}{1 + K_{\square}(SRT)} + \frac{QY_n(NO_x)}{1 + K_{dn}(SRT)} + Q(nbVSS)\ \ Equation\ 8.15\ of\ M\ \&\ E$$

$$P_{X,VSS} = A + B + C + \text{D}$$

$$= Heterotrophic\ biomass + Cell\ debris + Nitrifying\ bacteria\ biomass + Nonbiodegradable\ VSS\ in\ influent$$

The term 'C' is the biomass due to Nitrogen. If 'N' is ignored, then term 'C' is dropped.

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As per section 7.6 of Metcalf and Eddy, 'The total mass of dry solids wasted per day depends on the TSS, which includes the VSS plus inorganic solids. Inorganic solids are in the influent wastewater (TSS – VSS) and the biomass contains 10 to 15 percent inorganic solids by dry weight. Equation 8.15 is modified to calculate the solids production in terms of TSS by assuming a typical biomass VSS/TSS ratio of 0.85. The ratio of VSS/TSS varies from 0.8 to 0.9'. Further, inorganic solids in the influent wastewater $(TSS_o - VSS_o)$ contribute to inorganic solids and are additional solids production term that must be added to equation 8.15 to produce equation 8.16.

Influent solids

$$P_{X,TSS} = \frac{A}{0.85} + \frac{B}{0.85} + \frac{C}{0.85} + D + Q(TSS_o - VSS_o)$$

$Equation\ 8.16\ of\ Metcalf\ and\ Eddy$

Where

$P_{X,TSS}$ = net waste activated sludge produced per day, measured in terms of total suspended solids, Kg/day

TSS_o = Influent wastewater TSS concentration, g/m^3

VSS_o = Influent waste water VSS concentration, g/m^3

If 'N' is ignored in the design, the term 'C' is dropped from equation 8-16.

$$P_{X,TSS}(SRT) = \frac{QY(S_o - S)(SRT)}{\left(1 + K_d(SRT)\right)(0.85)} + \frac{f_d k_d Y Q(S_o - S)(SRT)^2}{\left(1 + K_d(SRT)\right)(0.85)} + \frac{QY_n(NO_x)(SRT)}{\left(1 + K_{dn}(SRT)\right)(0.85)} + Q(nbVSS)(SRT) + Q(TSS_o - VSS_o)SRT$$

Develop input data to solve Equation 8.16 of Metcalf and Eddy.

nbVSS = 45.675 mg/l

Assume $S_o = S_o - S$

S_o = bCOD = 400 mg/l

Q = 6000 / 2 = 3000 m^3/ day per tank

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$iTSS_o$ = 112.5 mg/l

Adjust the Kinetic Coefficients give in Table 1.1 and Table 1.2 to 14.5^o C.

Y = 0.4 g VSS / g bCOD

$K_{d,14.5^oC}$ = 0.0967 g VSS / g VSS. Day

f_d = 0.15 g/g

Substitute the above values in Equation 8.16, part by part.

70. A = $\frac{QY(S_o - S)(SRT)}{\left(1+K_d(SRT)\right)(0.85)} = \frac{3000(0.4)(400)(SRT)}{\left(1+0.0967(SRT)\right)x0.85} = \frac{564705.882(SRT)}{(1+0.0967(SRT))}$

71. B = $\frac{f_d k_d YQ(S_o - S)(SRT)^2}{\left(1+K_d(SRT)\right)(0.85)} = \frac{(0.15)(0.0967)(0.4)(3000)(400)(SRT)^2}{\left(1+0.0967(SRT)\right)x0.85} =$

$\frac{8191.059(SRT)^2}{\left(1+0.0967(SRT)\right)}$

72. D = $Q(nbVSS)(SRT)$ = 3000 x (45.675)(SRT) = 137025 (SRT)

73. E = $Q(TSS_o - VSS_o)SRT$ = 3000 x (375-262.5)SRT = 337500(SRT)

74. 8750000 = A + B + D + E

$$8750000 = \frac{564705.882}{(1 + 0.0967(SRT))} + \frac{8191.059}{\left(1 + 0.0967(SRT)\right)} + 137025(SRT) + 337500(SRT)$$

Solve for SRT by trial and error to get

SRT = 11.06 days.

As per Table 5.57 of CPHEEO manual, the recommended SRT lies between 4 – 20 days. Hence the obtained value of 11.06 days is OK.

What if Analysis of MS Excel:

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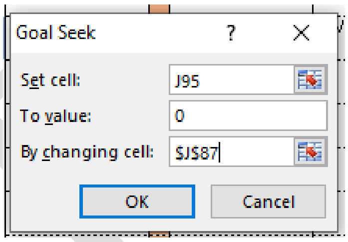
The 'What If' Analysis function of MS Excel is used to solve algebraic equations by trial and error.

i. Cell 73J contains the expression A + B + D + E - ($P_{X,TSS}$) SRT i.e. A + B + D + E – 8750000, which is in terms of SRT.

ii. In order to solve it, go to 'DATA' – 'What If Analysis' - ' Goal Seek' in the Menu bar of MS Excel.



iii. Place the cursor on cell 'J 95' and click 'goal seek'. Enter value of 'Set Cell:' , 'To value:' '0' by changing cell 'J87' and click on 'OK'. The calculated SRT will appear in cell J87.

Determine MLVSS concentration in the reactor tank

The MLVSS concentration is obtained by solving equation 7-54 of Metcalf and Eddy.
Mass of MLVSS = (X_{MLVSS}) V_T = ($P_{X,VSS}$) SRT -- Equation 7-54 of Metcalf and Eddy.
($P_{X,VSS}$)(SRT) is given by Equation 8-15 of Metcalf and Eddy.

$$P_{X,VSS} = \frac{QY(S_o - S)}{(1 + K_d(SRT))} + \frac{f_d k_d YQ(S_o - S)(SRT)}{(1 + K_d(SRT))} + \frac{QY_n(NO_x)}{(1 + K_{dn}(SRT))} + Q(nbVSS)$$

$Equation\ 8 - 15\ of\ Metcalf\ and\ Eddy$

Where
$P_{X,VSS}$ = Net waste activated sludge produced each day, Kg VSS/day
S_o = Influent substrate concentration, mg/l
S = effluent substrate concertation, mg/l

Equation 8-15 can be modified as below by multiplying both sides with (SRT).

$$P_{X,VSS}(SRT) = \frac{QY(S_o - S)(SRT)}{(1 + K_d(SRT))} + \frac{f_d k_d YQ(S_o - S)(SRT)^2}{(1 + K_d(SRT))} + \frac{QY_n(NO_x)(SRT)}{(1 + K_{dn}(SRT))} + Q(nbVSS)(SRT)$$

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This equation give the amount of VSS wasted per day per tank

Substitute the relevant values

SRT = 11.06 days

Assume $S_o = S_o - S$

S_o = bCOD = 400 mg/l

75. $A = \frac{QY(S_o - S)(SRT)}{(1+K_d(SRT))} = \frac{3000(0.4)(400)(11.06)}{(1+0.0967(11.06))} = 2564733.73$

76. $B = \frac{f_d k_d YQ(S_o - S)(SRT)^2}{(1+K_d(SRT))} = \frac{(0.15)(0.0967)(0.4)(3000)(400)(11.06)^2}{(1+(0.0967)(11.06))} = 411415.192$

77. $D = Q(nbVSS)(SRT)$ = 3000 x 45.726 x 11.06 = 1516811.215

78. $P_{X,VSS}(SRT) = \frac{3000(0.4)(400)(11.06)}{(1+0.0967(11.06))} + \frac{(0.15)(0.0967)(0.4)(3000)(400)(11.06)^2}{(1+(0.0967)(11.06))} +$

$3000(45.726)(11.06)$

$P_{X,VSS}(SRT) = 4492960.134\ g/day$ = 4492.960 Kg/ day

79. $(X_{MLVSS})\ V_T = 4492960.134\ g/day$

(X_{MLVSS}) =4492960.134 /2500 = 1797.184 g/m^3

80. Determine the fraction of MLVSS

$$\frac{X_{MLVSS}}{X_{MLSS}} = \frac{1797.184}{3500} = 0.513$$

Determine Biomass as VSS wasted per day

For the case where nitrification is not considered, as per Metcalf and Eddy, '$P_{X,BIO}$ includes the active biomass and cell debris derived from cell growth and is thus the sum of terms A and B of Equation 7-52.'

$$P_{X,BIO} = \frac{QY(S_o - S)}{(1+K_d(SRT))} + \frac{f_d k_d YQ(S_o - S)(SRT)}{(1+K_d(SRT))}$$ Equation 7-52 of Metcalf and Eddy

$P_{X,BIO} = A + B$

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81. A = $\frac{QY(S_o - S)}{(1+K_d(SRT))} = \frac{3000(0.4)(400)}{(1+0.0967(11.06))}$ = 231949.467

82. $B = \frac{f_d k_d YQ(S_o - S)(SRT)}{(1+K_d(SRT))} = \frac{0.15(0.0967)(0.4)(3000)(400)(11.06)}{(1+0.0967(11.06))}$ = 37207.58

83. $P_{X,BIO}$ = A + B = 213949.467 + 37207.58 = 269157.047 g / day

84. $P_{X,BIO}$ = 269157.047 / 1000 = 269.157 Kg / day

Determine the Decant Pumping Rate

85. Decant pumping rate = filling volume / decant time = 750 / 0.5 = 1500 m^3/hour
= 1500 / 60 = 25 m^3 / minute

Determine the Oxygen required per tank

The oxygen required is given by equation 8-17 of Metcalf and Eddy.
Ro = Q (So – S) – 1.42 PX,bio + 4.33 Q NOx   Equation 8-17 of Metcalf and Eddy
The term 4.33 Q NOx is dropped from the equation when Nitrogen is not considered.
Ro = Q (So – S) – 1.42 PX,bio

86. Ro = 3000 x 400/1000 – 1.42 x 267.157 = 817.797 Kg / day / tank

87. Determine the total aeration time
Aeration time per / cycle = 3.0 hours
Number of cycles / day = 4
Total aeration time = 3.0 x 4 = 12 hours / day

88. Average oxygen transfer rate = 817.797 / 12 = 68.150 Kg / hour

89. Factor for higher oxygen in the beginning = 1.5

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As per Metcalf and Eddy, 'The oxygen demand will be higher at the beginning of the aeration period. This higher demand is accounted for by multiplying the demand by a factor of 1.5 to 2 to provide sufficient oxygen transfer at the beginning of the cycle.'

90. Practical average oxygen transfer rate = 68.150 x 1.5 = 102.225 Kg / hour

Determine Sludge Production

91. The sludge production is determined by using Equation 7-55 of Metcalf and Eddy.
$P_{X,TSS}$ = V (MLSS) / SRT
= (2 x 2500) x 3500 / (11.06 x 1000) = 1582.666 Kg/day

92. bCOD removal rate = Design flow x bCOD = 6000 x 400 / 1000 = 2400 Kg / day

93. BOD removal rate = bCOD removal rate / 1.6 = 2400 / 1.6 = 1500 Kg / day

94. TSS observed yield = g TSS / g BOD = 1582.666 / 1500 = 1.055 g TSS/ g BOD

95. VSS observed yield = g VSS / g BOD = TSS observed yield x Fraction of MLVSS
= 1.055 x 0.513 = 0.5417 g VSS / g TSS

96. TSS/bCOD observed yield = 1582.666 / 2400 = 0.659 g TSS / g bCOD

97. Sludge wasting rate from full tank at MLSS concentration = $P_{X,TSS}$ / X_{MLVSS}
1582.666 / (3500 / 1000) = 452.190 m^3 / day

98. Sludge wasting from settled sludge = $P_{X,TSS}$ / X_S based on SVI value
= 1582.666 / (6666.667/100) = 237.40 m^3 / day

Determine the F/M ratio and BOD Volumetric Loading

99. F / M = Q x BOD / (XMLVSS x V)
= (6000 /2) x 250 / (1797.184 x 2500) = 0.167

100. BOD volumetric loading = Kg BOD / m^3.day = Q x BOD / V
= 6000 x 250 / (2 x 2500 x 1000) = 0.30

Determine Airflow Rate as per Rule of Thumb

As per Metcalf and Eddy, ‘ As an approximation, for BOD removal only, the oxygen requirement will vary from 0.9 to 1.3 kg O2 per Kg BOD removed for SRTs of 5 to 20 days, respectively.’

101. Oxygen requirement as per rule of thumb for removal of BOD
Design flow x (BOD_{IN} – BOD_{OUT}) x Oxygen needed per Kg BOD / 24 hours
= 6000 x (250 – 30) x 1.1 / (24 x 1000) = 60.5 kg / hour

102. Obtain the Standard Oxygen Transfer efficiency (SOTE)
The SOTE value is extracted from the catalogue of diffuser manufacturer. In the present case:
SOTE as a function of depth is = 6.56% per meter depth.

The tank depth = 6.0 m
Let the level of diffusers be 60 cm above the tank bottom.
Hence depth of diffusers below the free surface = 6.0 – 0.6 = 5.4 m

SOTE = 6.56 x 5.4 = 35.424%

103. Obtain the actual oxygen transfer efficiency (AOTE)
AOTE = SOTE x AOTE/SOTE ratio
As per diffuser data from manufacturer, AOTE / SOTE = 0.33
Hence AOTE = 0.33 x 35.424 = 11.69%

104. Required average oxygen transfer rate (AOTR) during aeration period (per tank)
= (oxygen requirement/No. of tanks) x (Tc / t_A) = (60.5 / 2) x (6 / 2) = 90.75 Kg/hour

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105. Air requirement = Oxygen required / (AOTE/1000) / Oxygen content in air
As per standard atmospheric data, oxygen content in air is 0.293 Kg/m^3
Air requirement = 90.75 / ((11.69/100) x 0.293) = 2649.52166 N m^3 / hour

106. Factor for increase of oxygen transfer rate at the beginning of the cycle = 1.5
Reference: 'The oxygen transfer rate should be multiplied by a factor of 1.5 to 2.0 to provide sufficient oxygen transfer at the beginning of the cycle' – Metcalf and Eddy.

107. Design air flow rate taking into account the factor for increase = 1.5 x 2649.82166
= 3974.28249 Normal m^3/hour

108. Blower outlet pressure = 1.6 bar
The blower outlet pressure can be compared with the value obtained from exact calculation using biological kinetics.

Determine air flow rate as per Biological Kinetics using Diffusers

The following calculations are adopted form the design of Activated Sludge Process in Metcalf and Eddy. Determination of airflow rate as per biological kinetics is not covered in Metcalf and Eddy under SBR design.

109. Average oxygen transfer rate per tank = R_o / 24 = 817.797 / 24 = 34.075 Kg/hour/tank

110. Required average oxygen transfer rate during aeration, AOTR = Average oxygen transfer rate per tank / (Total cycle time / Fill period) = 34.075 / (6/2)
= 102.22462 Kg/hour/tank.
This value is the same as already obtained in step (89) above.

Bubble Aeration Design

The actual amount of oxygen required (AOTR) must be obtained by applying correction factors to standards oxygen requirements (SOTR). The corrections factors compensate for effects of salinity-surface tension, temperature, elevation, diffused depth, desired oxygen operating level, and effects of mixing intensity and basin configuration. Equation 5-55 of Metcalf and Eddy takes into account all these correction factors.

Actual Oxygen Transfer Rate (AOTR) is obtained using equation 5-55 of Metcalf and Eddy:

$SOTR = AOTR\left[\frac{C_{S,20}}{\alpha F(\beta C_{\bar{S},T,H} - C_L)}\right](1.024^{(20-T)})$ Equation 5-55 of Metcalf and Eddy

Where

SOTR = standard oxygen transfer rate in tap water at 20° C and zero dissolved oxygen, Kg O_2 / hour

AOTR = actual oxygen transfer rate under field conditions, Kg O_2 / hour

$C_{S,20}$ = D.O. saturation concentration in clean water at 20° C and 1 atmosphere pressure, mg/l

α = Oxygen transfer correction factor for waste. Varies from 0.3 to 1.2.

F = Fouling factor, typically 0.65 to 0.9

β = Salinity-surface tension correction factor, typically 0.70 to 0.98

$C_{\bar{S},T,H}$ = Average dissolved oxygen saturation concentration in clean water in aeration tank at temperature T and altitude H, mg/l

C_L = operating oxygen concentration in the tank, mg/l

111. Compute P_b / P_a

As per Appendix B2 of Metcalf and Eddy, the value of P_b / P_a is used to compute the change in atmospheric pressure with elevation

$$\frac{P_b}{P_a} = e^{-\frac{gM(Z_b - Z_a)}{RT}}$$

g = 9.81m/s^2

M = mole of air = 28.97 Kg/Kg-mole

Z = elevation in m

R = Universal gas constant = 8314 N.m/Kg.mole.K

T = Temperature in Kelvin = (273.15 + ° C)

In the present study, Z = 310 m and Temperature = 14.5°C

$$\frac{P_b}{P_a} = e^{-\frac{9.81(28.97)(310-0)}{(8314)(273.15+14.5)}} = 0.96383$$

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112. Oxygen concentration at design temperature and altitude is obtained by applying the above correction factor.

$C_{14.5^oC}$ = 10.18 mg/l

The value is obtained from Table D1 of Appendix D of Metcalf and Eddy for zero salinity. If salinity is known, the same table can be used to obtain the relevant C value.

$C_{S,T,H}$ = 10.18 x 0.96383 = 9.81181 mg/l

113. Obtain the atmospheric pressure in m of water at design temperature and altitude by using the data in Appendix B and C of Metcalf and Eddy.

$$P_{atm,H} = \frac{\frac{P_b}{P_a}(P_{standard\ atm\ pressure})}{\gamma_{design\ T}}$$

The specific weight of water at 14.5^o C temperature is obtained from Table C-1 of Metcalf and Eddy by interpolation.

$\gamma_{water\ at\ design\ T}$ = 9.7986 KN/m^3 at a temperature of 14.5^o C

Standard atmospheric pressure = 101.325 KN/m^2

$$P_{atm,H} = \frac{0.9638(101.325)}{9.7986} = 9.96675\ m\ of\ water$$

114. Determine $C_{\bar{S},T,H}$ using equation from Metcalf and Eddy

$$C_{\bar{S},T,H} = C_{S,T,H}(\frac{1}{2})(\frac{P_d}{P_{atm,H}} + \frac{O_t}{21})$$

Where

P_d = pressure at the depth of air release, in KPa

$P_{atm,H}$ = atmospheric pressure at altitude H, in KPa

O_t = Percent oxygen concentration leaving tank, usually 18 to 20 percent

Assume

O_t = 19% (Average of the range)

Also, depth of the aerators below water surface = 6.0 -0.6 = 5.4 m

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$$C_{\bar{S},T,H} = 9.8115\left(\frac{1}{2}\right)\left(\frac{9.966 + 5.4}{9.966} + \frac{19}{21}\right) = 12.0026\ mg/l$$

115. Determine the SOTR

$$SOTR = AOTR\left[\frac{C_{s,20}}{\alpha F\left(\beta C_{\bar{S},T,H} - C_L\right)}\right](1.024^{(20-T)})$$

From diffuser data given in Metcalf and Eddy:

α = Oxygen transfer correction factor for waste water = 0.70 (0.3 to 1.2)

β = Salinity-surface tension correction factor = 0.95 (0.95 to 0.98)

F = Fouling factor = 0.9 (0.65 to 0.9)

$$SOTR = 102.22462\left[\frac{9.08}{0.7(0.9)(0.95(12.0026 - 2)}\right](1.024^{(20-14.5)})$$

= 178.52893 Kg/hour

116. Density of air at design temperature

From Appendix B of Metcalf and Eddy, obtain the density of air at 14.5^{o}C

ρ = P x M/(R xT)

Where

P_{atm} = atmospheric pressure = 101.325 KPa

P = 0.9638 x 101.325 = 97.657 KPa

M = mole of air = 28.97 Kg/Kg mole

R = universal gas constrant = 8314 J / Kg.K

T = temperature in K = 273.15 + ^{o}C

ρ = P x M/(R xT) = 97.657 x 10^3 x 28.97 / (8314 x (273.15 + 14.5))

= 1.18302 Kg/m^3

117. Air contains 23.18% oxygen by weight.

Corresponding amount of oxygen by weight = 0.2318 x 1.183 = 0.27422 Kg/m^3

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118. Estimate the air flow rate corresponding to above SOTR

Air flowrate = SOTR / (E x 60 min/hour)

Where E = efficiency of fine bubble ceramic diffusers with an aeration clean water transfer efficiency of 35%

Air flowrate = 178.52893 / (0.35 x 60 x 0.27422) = 31.00159 m^3/min

119. Air flowrate = 31.00159 x 60 = 1860.09563 m^3 / hour

120. Factor for increase of oxygen transfer rate at the beginning of the cycle = 1.5

Reference: 'The oxygen transfer rate should be multiplied by a factor of 1.5 to 2.0 to provide sufficient oxygen transfer at the beginning of the cycle' – Metcalf and Eddy.

121. Design air flow rate taking into account the factor for increase = 1.5 x 1860.09563 = 2790.14344 Normal m^3/hour

122. Blower outlet pressure = 1.6 bar

Diffuser Data

123 to 139: Data related to diffuser design and related items is taken from Metcalf and Eddy

Table 2. 26 Summary of design of SBR reactor

Component	Capacity		
	1 MLD`	**2 MLD**	**5 MLD**
SBR Reactor details			
No. of SBR tanks	2	2	2
L (m)	14.5	20.5	32.5
B (m)	14.5	20.5	32.5
D (m)	6.3	6.3	6.3

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Aeration required			
Nm^3/hour	1395.07	2790.14	6975.36

2.9 Design of Extended Aeration Process

The extended aeration process is one of the modifications of the Activated Sludge Process. It is a complete mix system and provides biological treatment for the removal of biodegradable organic wastes under aerobic conditions. As the name suggests, aeration is carried out for a prolonged period leading to complete stabilization of the sludge in the aeration tank, thereby eliminating the requirement of sludge digester. The suspended solids in raw sewage also settle down in the aeration tank and get digested. Air may be supplied through surface aerators or diffused aeration to provide the oxygen required to sustain the aerobic biological process.

The extended aeration plants are easy to operate, the process is odour free and the footprint is relatively small compared to other technologies. Due to prolonged detention, the system is better suited to handle inflow fluctuations and variable organic loadings. A major advantage is the low sludge yield and the sludge is fairly stabilized so that it can be directly sent to drying bed. The main disadvantage of the system is that it requires more energy and skilled personnel to maintain the MLSS concentration by regulating the return sludge.

Figure 2.15 shows the process flow diagram of extended aeration system. In general the hydraulic retention time is close to 24 hours and another 4 hours are required in the secondary sedimentation tank.

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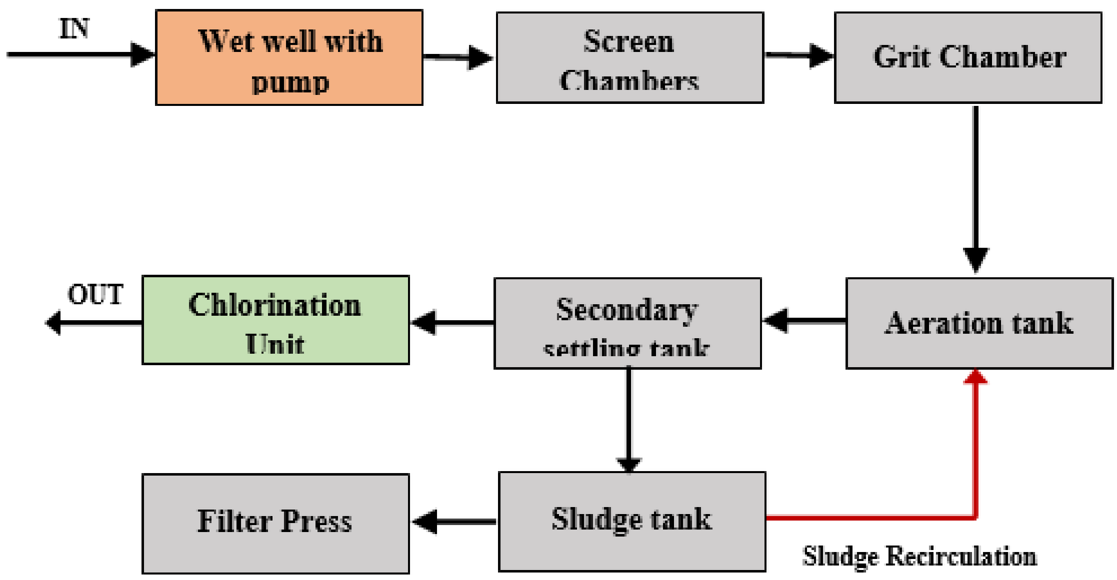

Figure 2. 15 Flowchart showing the conceptualization of extended aeration plant

2.9.1. Working of Extended Aeration System

Primary treatment consisting of screens and grit chamber are necessary in the extended aeration system. After primary treatment, the flow is directed to the aeration tank which constitutes the secondary treatment. In aeration tank the wastewater is mixed with activated sludge and oxygen is provided to the microorganisms through diffused aeration or surface aerators. The waste water is retained in the aeration tank for about 24 hours in order to decompose organic matter present in the waste water. The mixed liquor then flows to a secondary settling tank (SST) where most microorganisms settle to the bottom of the settling tank along with sludge. In order to maintain the MLSS in the aeration tank, about 50% of the sludge is pumped back to the aeration tank to mix with incoming raw sewage and thereby ensure sufficient microorganisms. The clarified wastewater from the SST then flows over V-notches into the effluent launder and into the chlorination tank for final treatment.

The following extract from the CPHEEO manual underlines the basic design principle of the extended aeration system.

"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, the MLSS undergo considerable endogenous respiration and get well stabilized and in these cases, the

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excess sludge does not require separate digestion and it can be directly dried on sand beds or dewatered in the equipment. 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."

The typical values of design parameters, extracted from 'Wastewater Engineering, Treatment and Reuse by Metcalf and Eddy, 4th edition, 2003 are presented in Table 2.27.

Table 2. 27 Typical range of design parameters for extended aeration process

Activated Sludge Process	Volumetric Loading		F:M	HRT hours
	lb BOD/day / 1000 ft^3	kg BOD/day / m^3	lb BOD/day / lb MLVSS	
Conventional Plug Flow	20 -40	0.3 - 0.7	0.2 - 0.4	4 - 8
Complete Mix	20 - 100	0.3 - 1.6	0.2 - 0.6	3 - 5
Extended Aeration	5 - 15	0.1 - 0.3	0.04 - 0.1	20 - 30

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2.9.2 Design Calculations of Extended Aeration Process

The design calculations for 1MLD, 2 MLD and 5 MLD extended aeration process are presented in Table 2.28.

Table 2. 238 Design Calculations of 1, 2 and 5 MLD Extended Aeration Reactor

DESIGN OF 1 MLD, 2 MLD AND 5 MLD STP BASED ON EXTENDED AERATION PROCESS

S. No.	Item	1 MLD		2 MLD		5 MLD	
		Value	Unit	Value	Unit	Value	Unit
	INPUT DATA AND SEWAGE CHARACTERIRTICS						
1	Average design flow into the STP	1.00	MLD	2.00	MLD	5.00	MLD
2	Average design flow into the STP	1000.00	m^3/day	2000.00	m^3/day	5000.00	m^3/day

3	Average design flow into the STP	41.67	m^3/hour		83.33	m^3/hour		208.33	m^3/hour
4	Average design flow into the STP	0.0116	m^3/s		0.0231	m^3/s		0.0579	m^3/s
5	Elevation of site above MSL	310.00	m		310.00	m		310.00	m
6	Operating temperature	14.50	ºC		14.50	ºC		14.50	ºC
7	Influent wastewater BOD	250.00	mg/l		250.00	mg/l		250.00	mg/l
	EFFLUENT STANDARDS								
8	Effluent BOD	30.00	mg/l		30.00	mg/l		30.00	mg/l
9	Effluent COD	250.00	mg/l		250.00	mg/l		250.00	mg/l
10	Effluent TSS	100.00	mg/l		100.00	mg/l		100.00	mg/l
	DESIGN PARAMETERS AND ASSUMPTIONS								
11	Thickener overflow return as fraction of plant flow	0.15	Dimensionless		0.15	Dimensionless		0.15	Dimensionless
12	Thickener overflow return	0.15	MLD		0.30	MLD		0.75	MLD
13	Thickener overflow return BOD	500.00	mg/l		500.00	mg/l		500.00	mg/l
14	Centrate from sludge dewatering as fraction of plant flow	0.006			0.006			0.006	
15	Centrate from sludge dewatering return	0.006	MLD		0.012	MLD		0.030	MLD
16	Centrate from sludge dewatering return BOD	380.00	mg/l		380.00	mg/l		380.00	mg/l
17	Influent BOD to aeration tank	283.11	mg/l		283.11	mg/l		283.11	mg/l
18	Weighted BOD to be removed in the aeration tank	253.11	mg/l		253.11	mg/l		253.11	mg/l

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19	MLSS	4500.00	mg/l		4500.00	mg/l		4500.00	mg/l
20	F : M Ratio	0.15	1/day		0.15	1/day		0.15	1/day
21	Total volatile fraction (MLVSS) of MLSS	0.60			0.60			0.60	
	DETERMINATION OF REACTOR VOLUME								
	Required Reactor Volume based on F:M ratio								
22	F	292.60	Kg/day		585.20	Kg/day		1463.00	Kg/day
23	M	1950.67	Kg		3901.33	Kg		9753.33	Kg
24	Aeration tank volume calculated from F/M using Equation 5.27 of CPHEEO Manual	722.47	m^3		1444.94	m^3		3612.34	m^3
	Required Reactor Volume based on SRT								
25	Mean Cell Residence Time or SRT, θ_c as per Figure 5.38 of CPHEEO Manual	25.00	days		25.00	days		25.00	days
26	Constant Y	0.50			0.50			0.50	
27	constant Kd	0.06	1/day		0.06	1/day		0.06	1/day
28	Aeration tank volume calculated from SRT using Equations 5.24 and 5.25 of CPHEEO Manual	281.24	m^3		562.48	m^3		1406.19	m^3
	Required Reactor Volume Based on HRT								
29	HRT for average flow as per Table 5.9 of CPHEEO manual	24.00	hours		24.00	hours		24.00	hours
30	Aeration tank volume calculated from HRT	1000.00	m^3		2000.00	m^3		5000.00	m^3
31	Maximum of the three volumes of aeration tank, cum	1000.00	m^3		2000.00	m^3		5000.00	m^3
32	Volumetric BOD loading rate	0.25	Kg BOD / day / m^3		0.25	Kg BOD / day / m^3		0.25	Kg BOD / day / m^3

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	SIZING OF THE REACTOR								
33	Depth of liquid in the reactor tank	5.50	m		5.50	m		5.50	m
34	L/B ratio	1.00	m		1.00	m		1.00	m
35	Length of the tank	13.48	m		19.07	m		30.15	m
36	Width of the tank	13.48	m		19.07	m		30.15	m
37	Adopted length of the tank	13.50	m		19.25	m		30.25	m
38	Adopted width of the tank	13.50	m		19.25	m		30.25	m
	DETERMINATION OF AERATION REQUIREMENT								
39	BOD removed in aeration tank	292.60	Kg/day		585.20	Kg/day		1463.00	Kg/day
40	Oxygen requirement at Kg oxygen / Kg of BOD removed	1.10	Kg O_2/Kg BOD		1.10	Kg O_2/Kg BOD		1.10	Kg O_2/Kg BOD
41	Kg of Oxygen needed per day	321.86	Kg O_2 per day		643.72	Kg O_2 per day		1609.30	Kg O_2 per day
42	Residual D. O. in aeration tank	2.00	mg/l		2.00	mg/l		2.00	mg/l
	CAPACITY OF SURFACE AERATORS								
43	α, ratio of oxygen uptake rate of sewage to that of clean tap water at 20°C	0.83			0.83			0.83	
44	β, multiplying factor for dissolved oxygen saturation for sewage at operating temperature	0.95			0.95			0.95	
45	D O at operating temperature	10.37	mg/l		10.37	mg/l		10.37	mg/l
46	D O at operating elevation	10.01	mg/l		10.01	mg/l		10.01	mg/l
47	Oxygen tension, mg/l	7.51	mg/l		7.51	mg/l		7.51	mg/l

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48	Oxygen gradient, mg/l	0.82	mg/l		0.82	mg/l		0.82	mg/l
49	Temperature difference	-5.50	°C		-5.50	°C		-5.50	°C
50	Temperature Co-efficient	1.02			1.02			1.02	
51	Temperature correction factor	0.88			0.88			0.88	
52	Conversion factor to standard conditions	0.60			0.60			0.60	
53	Oxygen needed under standard conditions, kg / day	539.66	Kg/day		1079.31	Kg/day		2698.28	Kg/day
54	Provide factor of safety for intangibles	1.10			1.10			1.10	
55	Oxygen needed after factor of safety	593.62	Kg/day		1187.24	Kg/day		2968.11	Kg/day
56	Oxygen transfer capacity of aerator	1.80	Kg / KWh		1.80	Kg / KWh		1.80	Kg / KWh
57	Required Aerator Capacity	13.74	KW		27.48	KW		68.71	KW
	CAPACITY OF DIFFUSED AERATORS								
58	Standard temperature	20.0	°C		20.0	°C		20.0	°C
59	Density of air at operating temperature	1.24	Kg/m^3		1.24	Kg/m^3		1.24	Kg/m^3
60	Density of air at 20°C temperature	1.23	Kg/m^3		1.23	Kg/m^3		1.23	Kg/m^3
61	Content of oxygen in air	0.23			0.23			0.23	
62	Kg of oxygen needed for residual D O per day	2.00	Kg / 2 mg/l O_2		4.00	Kg / 2 mg/l O_2		10.00	Kg / 2 mg/l O_2
63	Total kg of oxygen needed per day	323.86	Kg/day		647.72	Kg/day		1619.30	Kg/day
64	cum of air needed per day	1121.92	m^3/day		2243.84	m^3/day		5609.61	m^3/day
65	Transfer efficiency of diffuser system per m depth	0.05			0.05			0.05	

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66	Transfer efficiency at design depth	0.28			0.28			0.28	
67	diffuser fouling factor per year	0.04	1/year		0.04	1/year		0.04	1/year
68	Diffuser life cycle	3.00	years		3.00	years		3.00	years
69	Diffuser fouling factor for its life cycle	1.12			1.12			1.12	
70	Provide factor of safety for intangibles	1.10			1.10			1.10	
71	Air needed for oxygenation	5048.03	m^3/day		10096.07	m^3/day		25240.17	m^3/day
72	Air needed for oxygenation in cum / hour	210.33	m^3/hour		420.70	m^3/hour		1051.67	m^3/hour
73	Air mixing criteria cum /minute / 1000 cu m of tank	16.00	m3/hour/1000 cum		16.00	m3/hour/1000 cum		16.00	m3/hour/1000 cum
74	Air needed for mixing as per manual cum / hr	960.00	m^3/hour		1920.00	m^3/hour		4800.00	m^3/hour
75	Minimum air needed for mixing as per manual	2.70	m^3/hour/m^2		2.70	m^3/hour/m^2		2.70	m^3/hour/m^2
76	Surface area of aeration tank	181.82	m^2		363.64	m^2		909.09	m^2
77	Air needed for mixing as per US EPA guidelines	490.91	m^3/hour		981.82	m^3/hour		2454.54	m^3/hour
78	Adopted value of air needed for mixing	960.00	m^3/hour		1920.00	m^3/hour		4800.00	m^3/hour
79	Air needed as under standard conditions	1609.61	m^3/hour		3219.23	m^3/hour		8048.07	m^3/hour
80	Friction and other losses as fraction of depth	0.20			0.20			0.20	
81	Liquid depth as water column for air pressure	6.60	m		6.60	m		6.60	m
82	KW of needed compressor at 1400 rpm	47.96	KW		83.98	KW		192.05	KW
	SLUDGE PRODUCTION								
83	Y0bs=Y/(1+Kd*ThetaC)	0.20			0.20			0.20	
84	Excess Sludge mass wasted Kg/day = A	50.62	Kg/day		101.24	Kg/day		253.11	Kg/day

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85	Kg of excess sludge / Kg of BOD removed	0.40			0.40			0.40	
86	Kg of excess sludge from thumb rule per day	101.24	Kg/day		202.49	Kg/day		506.23	Kg/day
87	Excess sludge as higher of the two values	101.24	Kg/day		202.49	Kg/day		506.23	Kg/day
88	Concentration factor for MLSS in return / excess sludge	3.30			3.30			3.30	
89	Return / excess sludge MLSS concentration	14850.0	mg/l		14850.0	mg/l		14850.0	mg/l
90	Cells in aeration tank based on MLSS	4500.00	Kg		9000.00 0	Kg		22500.0 00	Kg
91	Cells wasting from system	101.24	Kg/day		202.49	Kg/day		506.23	Kg/day
92	Volume of excess sludge	6.82	m^3/day		13.64	m^3/day		34.09	m^3/day
93	Resulting SRT, θc	44.45	days		44.45	days		44.45	days
94	Least SRT, θc, in design	25.00	days		25.00	days		25.00	days
95	Volume of excess sludge for least θc	12.12	m^3/day		24.24	m^3/day		60.61	m^3/day
96	Excess sludge pump set duty as	12.12	m^3/day		24.24	m^3/day		60.64	m^3/day
97	Recirculation ratio	0.90			0.90			0.90	
98	Return sludge pump set duty as MLD	0.90	MLD		1.80	MLD		4.50	MLD

	Variable input
	Input from CPHEEO manual or other standards
	Output

2.9.2.1 Design Steps of 2 MLD Extended Aeration Process (EAP)

REFERENCES FOLLOWED

A. Manual on Sewerage and Sewage Treatment Systems, CPHEEO, November 2013.

B. Waste Water Engineering Treatment and Reuse, Metcalf and Eddy, 4th edition, Tata McGraw Hill edition.

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Input Data and Sewage Characteristics

1. Influent flow rate = Q_o = 2 MLD
   The design is being undertaken for a 2 MLD STP based on ASP technology.

2. Influent flow rate = 2000 m^3 /day
   MLD = 2 x 10^6 / 10^3 = 2000 m^3 /day

3. Influent flow rate = 83.3333 m^3 /hour
   2 MLD = 2000 / 24 = 83.3333 m^3 /hour

4. Influent flow rate = 2000 / (24 x 60 x 60) = 0.023148 m^3 /s

5. Elevation of the STP location = 310 m
   The STP is designed for a town in central India with a latitude of 21.14^o N. The average elevation of the town is 310 m as per topographic map.

6. Temperature = 14.5^{o}C
   In the present study, the plant is assumed to be located at a latitude of 21.14^o N (Central India) where the lowest winter temperature is obtained as 14.5^o C from the IMD records.

7. Influent BOD_5 = 250 mg/l
   As per the UASBR design, the BOD_5 of the effluent from the UASB reactor is 100 mg/l. Since the ASP reactor is in sequence with the UASB reactor, the influent BOD_5 into the ASP reactor shall be 100 mg/l

Effluent Standards

8. BOD_5 = 30 mg/l

9. COD = 250 mg/l

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10. TSS = 100 mg/l

With reference to Table 5.3 of the CPHEEO manual, the permitted level of BOD_5, COD, and TSS for discharge into inland surface water bodies is considered. The table is based on General standards for Discharge of Environmental Pollutants, Part A: Effluents. The present design is executed in order to meet the above mentioned limits.

Design Parameters and Assumptions

11. Sludge thickener overflow return into the reactor = 15%
Value assigned as per the input data in Appendix A 5-12 of CPHEEO Manual

12. Volume of thickener overflow return into the reactor
= Percentage return x average flow = 0.15 x 2.0 = 0.3 MLD

13. BOD of the thickener overflow return into the reactor = 500 mg/l

Value assigned as per the input data in Appendix A 5-12 of CPHEEO Manual

14. Centrate from sludge dewatering as fraction of plant flow = 0.6%
Value assigned as per the input data in Appendix A 5-12 of CPHEEO Manual

15. Volume of centrate from sludge dewatering
= centrate as fraction of plant flow x average flow = 0.006 x 2 =0.012 MLD

16. BOD of the centrate from sludge dewatering = 380 mg/l

Value assigned as per the input data in Appendix A 5-12 of CPHEEO Manual

17. Influent BOD into the aeration tank

This is the weighted BOD of the wastewater in the reactor, thickener overflow into the reactor and the centrate from sludge dewatering.

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Influent BOD in to the aeration tank = $= \frac{(2.0\ x\ 250)+(0.30x500)+(0.012x380)}{(2.0+0.30+0.012)}$

= 283.1142 mg/l

18. Weighted BOD to be removed from the aeration tank = Influent BOD – Effluent BOD = 283.1142 – 30 = 253.1142 mg/l.

19. MLSS = 4500 mg/l (Range is 3000 to 5000 mg/l)

In the extended aeration system, the MLSS is maintained on the higher side. The typical ranges of the values of operational parameters for extended aeration are given in Table 5.9 of the CPHEEO manual, which is presented below.

Table 5.9 from CPHEEO Manual

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

20. F : M Ratio = 0.15

The range of F : M is 0.1 to 0.18 from Table 5.9 of CPHEEO manual.

21. Total Volatile Fraction (MLVSS) of MLSS = 0.60

The MLSS, F:M ratio and the total volatile fraction is taken from the values furnished in Table 5.9 of CPHEEO manual.

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Determination of Reactor Volume

The capacity of the aeration tank is computed based on three criteria:

i. F : M ratio – 0.1 to 0.18 Kg BOD /day / Kg MLVSS
Adopted value = 0.15

ii. SRT – 10 to 26 days
Adopted value = 25 days

iii. Hydraulic retention time – 12 to 24 hours
Adopted value = 24 hours

Based on F: M ratio

22. Compute the F component of food to microorganism ratio (F:M)
F = Weighted BOD x (average flow + thickener overflow + centrate flow)
= 253.1142 x (2 + 0.3 + 0.012) = 585.2000 Kg/day

23. Compute the M component of food to microorganism ratio (F:M)
M = F/(M/F ratio) = 585.2000 / 0.15 = 3901.3333 Kg

24. Aeration tank volume = M x MLVSS fraction of MLSS / MLSS
= 3901.3333 / (0.6 x 4500) x 1000
= 1444.9383 m^3

Based on Solids Retention Time (SRT)

25. Mean cell residence time or SRT
θ_c = 25 days
The SRT value is obtained from Table 5.9 of CPHEEO manual.

26. Constant Y = 0.5

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The value of Y is taken from clause 5.8.1.3 of CPHEEO manual.

27. Constant K_d = 0.06 1/day

The value of K_d is taken from clause 5.8.1.3 of CPHEEO manual.

28. Required aeration tank volume based on SRT = 562.4760 m^3.

The calculation of the aeration tank volume is carried out as follows:

$$\theta_C = \frac{VX}{Q_W X_S} \; Equation\ 5.24\ of\ CPHEEO\ manual$$

$$Q_W = YQ(S_0 - S) - K_d V \;\; Equation\ 5.25\ of\ CPHEEO\ manual$$

Insert equation 5.25 into equation 5.24 and simplify to obtain the expression for V as:

$$V = \frac{YQ(S_0 - S)\theta_C}{X(1 + K_d\theta_C)}$$

$$V = \frac{0.5\ (2)(1000)(283.1142 - 30)(25)}{4500(1 + 0.06(25))} = 562.4760\ m^3$$

Based on HRT

29. Hydraulic Retention time (HRT) = 24.0 hours

The HRT time corresponds to the value prescribed in Table 5.9 of CPHEEO manual.

30. Aeration tank volume based on HRT = Q x 1000 x HRT/24

= 2 x 1000 x 24 / 24 = 2000.0000 m^3

31. Adopted volume = maximum volume among F:M ratio, SRT and HRT

= 2000.0000 m^3/s.

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32. From the above calculations it is observed that the required capacity of the aeration tank is 2000.0000 m^3. This produces a volumetric BOD loading rate of 0.2531 Kg BOD / day / m^3 which is within the prescribed limit as per Table 8-16 of Metcalf and Eddy.

i. $$Volumetric\ loading = \frac{BOD_{IN}(Average\ Design\ Flow)}{Volume\ of\ the\ tank} = \frac{253.1142(2000)}{(2000)(1000)} = 0.2531$$

This lies between the recommended value of 0.1 - 0.3 Kg BOD / day / m^3

Sizing of the Reactor

33. Depth of liquid in the reactor tank = 5.5 m (ASSUMED)

As per clause 5.8.1.7.5.1 in CPHEEO manual, 'The depth usually ranges from 3 m to 4.5 m for surface aerators. 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. In the present design the value of 5.5 m from Appendix A – 5.12 is adopted.

34. L/B Ratio = 1.0

As per clause 5.8.1.7.5.1 in CPHEEO manual, 'The width 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'. In light of this, he adopted L/B ratio is 1.0 is satisfactory.

35. Length of the tank = $\sqrt[2]{2000(1)/5.5 =}$ 19.0693 m.

36. Width of the tank = 1.0 x 19.0693 = 19.0693 m

37. Adopted length of the tank = 19.25 m

38. Adopted width of the tank = 19.25 m

Determination of Aeration Requirement

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39. BOD removed in the aeration tank = (Average flow + Thickener overflow + Centage from sludge dewatering) x weighted BOD
= (2.0 + 0.30 + 0.012) x 253.1124 = 585.2000 Kg/day

40. Oxygen requirement in Kg oxygen / Kg BOD removed.
This method employs rule of thumb in order to obtain the oxygen requirement. As per Table 5.9 of CPHEEO manual, the oxygen requirement is 1.0 to 1.2 Kg O_2/Kg BOD removed. In the present study the oxygen requirement is taken as 1.1 Kg O_2/Kg BOD removed.

41. Kg of oxygen needed per day = BOD removed per day x Oxygen required per Kg BOD
= 585.2000 x 1.1 = 643.7200 Kg O_2/day

42. Residual in aeration tank = 2 mg/l.
As per clause 5.8.1.7.5.3 of CPHEEO manual, '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.' Designers adopt 2 mg/l for both conventional and extended aeration plants. The same has been adopted in Appendix A-5.12 of CPHEEO manual.

Required Capacity of Surface Aerators

The oxygen transfer capacity of aerators under field conditions can be calculated from the standard oxygen transfer capacity at 20^o C and 760 mm Hg barometric pressure by using equation 5.30 of CPHEEO manual.

$$N = \frac{N_s(C_s - C_L)1.024^{T-20} \propto}{9.17}\ Equation\ 5.30\ of\ CPHEEO\ manual$$

Where

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N: Oxygen transferred under field conditions, Kg O_2 / KWh

N_s: Oxygen transfer capacity under saturated 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/l

T: Operating temperature, ^{o} C

α: Correction factor for oxygen transfer in sewage

43. Correction factor for oxygen transfer for sewage, α = 0.83.
α represents the ratio of the oxygen uptake rate, of the given sewage to that of clean tap water at 20^o C. As per clause 5.8.1.7.5.3 of CPHEEO manual, the value of α has to be chosen judiciously as it impacts the cost of the aeration system. The CPHEEO manual specifies a value of 0.8 to 0.85. Hence the chosen value of 0.83 is OK.

44. Multiplying factor for DO saturation for sewage at operating temperature, β = 0.95.
As per clause 5.8.1.7.5.3 of CPHEEO manual, the value of β is 0.95 for domestic sewage. Used for calculating C_s.

45. Dissolved oxygen at operating temperature = 10.3673 mg/l
Let operating temperature = T. As per Appendix A – 5.12 of CPHEEO manual,
D.O. at T = 14.42 + 0.003 x T^2 – 0.323 x T = 14.42 + 0.003 x 14.5^2 - 0.323 x 14.5
= 10.3673 mg/l
The DO at 14.5^o C can also be obtained from Table 5.10 of CPHEEO manual.

46. DO at operating temperature and elevation = 10.0078 mg/l
Let site elevation = Z. As per Appendix A – 5.12 of CPHEEO manual,
DO at operating elevation = (1-(Z / 152) x 0.017) x DO at operating temperature
= (1 – (310 / 152) x 0.017) x 10.3673 = 10.0078 mg/l.
The DO at 310 m elevation can also be obtained from Table 5.11 of CPHEEO manual.

47. Oxygen Tension = (Cs –C_L) = 7.5074
As per Appendix A-5.12 of CPHEEO manual the oxygen tension is given by:
C_s x β – Residual oxygen in aerator.
Oxygen Tension = (Cs –C_L) = 10.0078 x 0.95 – 2.0 = 7.5074 mg/l

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48. Oxygen gradient = 0.8187 mg/l
    As per Appendix A-5.12 of CPHEEO manual, Oxygen gradient = (C_s –C_L)/9.17
    = 7.5074 / 9.17 = 0.8187 mg/l

49. Temperature difference = operating temperature – standard temperature
    T – 20 = 14.5-20 = -5.5ºC

50. Temperature coefficient = 1.024
    This is as per equation 5.30 of CPHEEO manual.

51. Temperature correction factor
    = $1.024^{(T-20)}$ = $1.024^{-5.5}$ = 0.8777

52. Conversion factor to standard conditions
    $$= \frac{(C_s - C_L)1.024^{T-20} \propto}{9.17} \; Equation\ 5.30\ of\ CPHEEO\ manual$$
    = 0.8187 x 0.8777 x 0.83 = 0.5964

53. Oxygen needed under standard conditions = Kg of oxygen needed per day / conversion factor = 643.72 / 0.5964 = 1079.3133 Kg/day

54. Factor of safety for aeration intangibles = 1.1
    The 10% increase is in accordance to Appendix A – 5.12 of CPHEEO manual

55. Oxygen requirement after incorporating factor of safety as per Appendix – A 5.12 = Oxygen needed under normal conditions x Factor of safety
    = 1079.3133 x 1.1 = 1187.2446 Kg/day

56. Oxygen transfer capacity of aerator 1.8 Kg/KWh
    As per clause 5.8.1.7.5.3 of CPHEEO manual, the oxygen transfer capacity of aerator varies between 1.2 to 2.4 Kg/KWh. The average value is taken in the present study.

57. Required aerator capacity = 1187.2446 / (1.8 x 24) = 27.4825 KW

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Calculation of capacity of Diffused Aerators

The calculation is done as per clause 5.8.1.7.5.4 of CPHEEO manual, by further adjusting the air volume calculated based on the α value for surface aerators. This is necessary as the diffusers are located in the tank at 0.3 to 0.6 m above the tank floor and hence are subjected to a pressure of the overlying water column.

58. Standard Temperature = 20^{o} C
The standard temperature is used to compute the standard oxygen transfer rate (SOTR) from atmosphere to the wastewater.

59. Density of air at operating temperature is computed using an empirical equation given in Appendix A – 5.12 of CPHEEO manual.
$\rho_{T^{o}C} = 1.285 + T^3/10^6 – T^2/(10^5 \times 7) - 0.003 \times T$
$\rho_{14.5^{o}C} = 1.285 + 14.5^3/10^6 – 14.5^2/(10^5 \times 7) - 0.003 \times 14.5 = 1.2442$ Kg/m^3

60. Density of air at 20^{o} C temperature is computed using an empirical equation given in Appendix A – 5.12 of CPHEEO manual.
$\rho_{T^{o}C} = 1.285 + T^3/10^6 – T^2/(10^5 \times 7) - 0.003 \times T$
$\rho_{20^{o}C} = 1.285 + 20^3/10^6 – 20^2/(10^5 \times 7) - 0.003 \times 20 = 1.2324$ Kg/m^3

61. Content of oxygen in the air based on weight = 23. 20% = 0.2320

62. Kg of oxygen needed for residual DO of 2 mg/l per day = 2 x 2 = 4 Kg

63. Total oxygen needed per day = 643.72 + 4.0 = 647.72 Kg/day

64. Air needed = 647.72 /(0.2320 x 1.2442) = 2243.8420 m^3/day

65. Oxygen transfer efficiency of diffuser system per meter depth = 0.05
The oxygen transfer efficiency is specified by the manufacturer of the diffuser system and it is taken as 0.05 as per Appendix A – 5.12 of CPHEEO manual.

66. Transfer efficiency at design depth of 5.5 m = 5.5 x 0.05 = 0.275

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67. Diffuser fouling factor = 0.04

    As per clause 5.8.1.7.5.4 of CPHEEO manual, 'Fouling factor of diffusers at the rate of 4 % to 5 % per year over its life span.'

68. Diffuser life cycle = 3 year

    Adopted based on field observations and the specified value as per clause 5.8.1.7.5.4 of CPHEEO manual

69. Diffuser fouling factor, F, over its life cycle = $(1+0.04)^3$ = 1.1249

    The life cycle cost is obtained by using standard formula used in literature.

70. Factor of safety for intangible = FS = 1.1

    Taken as per clause 5.8.1.7.5.4 of CPHEEO manual.

71. Air needed for oxygenation in m^3/day = air needed x F x FS / Transfer efficiency at design depth = 2243.842 x 1.1249 x 1.1 / 0.2750 = 10096.0685 m^3/day
72. Air needed per hour = 10096.0685 / 24 = 420.6695 m^3/hour

73. Air mixing criteria = 16 m^3 / minute / 1000 m^3 of tank volume

    As per clause 5.8.1.7.5.6 in CPHEEO manual mentioned in Appendix A – 5.12
74. Air required for mixing as per criteria = Volume of aeration tank x 16 x 60 / 1000 = 2000.0000 x 16 * 60 / 1000 = 1920.0000 m^3/hour

75. Minimum air required volume required for mixing = 2.7 m^3/hour/m^2 of floor area

    Extracted from EPA 625/8-85/0100, p 38 and mentioned in clause 5.8.1.7.5.6 of CPHEEO manual.

76. Surface area of the aeration tank = Volume / depth = 2000.0000 / 5.5 = 363.6364 m^2

77. Volume of air needed as per US EPA guidelines = 363.6364 x 2.7 = 981.8182 m^3/hour

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78. Adopted value of air needed for mixing = Maximum of (1920.0000, 981.8182, 420.6695) = 1920.0000 m^3/hour

79. Requirement of air under standard conditions = Adopted requirement / Conversion factor = 1920.0000 / 0.5964 = 3219.2280 m^3/hour

80. Accounting for friction and other losses as fraction of depth = 0.2

81. Liquid depth for design of compressor capacity = depth x (1 + friction loss factor) = 5.5 x (1 + 0.2) = 6.6 m

82. Required capacity of the compressor at 1400 rpm

= 0.746 x ((0.03 x air volume/hour) + 16))
= 0.746 x ((0.03 x 3219.2280) + 16) = 83.9823 KW

As per Appendix A 5.12, for DPR purpose equation for compressor KW at 1400 rmp can be taken as:

i. For 7 m water column, BHP = 0.03*(cum / hr)+16

ii. For 6 m water column, BHP = 0.025*(cum / hr)+13

iii. For 5 m water column, BHP = 0.02*(cum / hr)+14

Sludge Production Calculations

83. As per clause 5.8.1.7.5.10 of CPHEEO manual, the excess sludge is obtained using the equation:

$$A = Q\, Y_{obs}(S_o - S)$$

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

Y = 0.5

K_d = 0.06

$$Y_{obs} = \frac{Y}{(1 + K_d\theta_c)} = \frac{0.5}{(1 + 0.06(25))} = 0.2000$$

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84. Excess sludge mass wasted = Y_{obs} x weighted BOD x average flow
= 0.2000 x 253.1142 x 2 = 101.2457 Kg/day

85. Ratio of excess sludge to BOD removed = 0.4
As per clause 5.8.1.7.5.10 of CPHEEO manual, '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. Hence the adopted value of 0.4 is OK.

86. Excess sludge from thumb rule = Q x Weighted BOD x Ratio of excess sludge to BOD
= 2 x 253.1142 x 0.4 = 202.4913 Kg/day

87. Adopted value of excess sludge = Higher of (84) and (86) = 202.4913 Kg/day

88. Concentration factor for MLSS in return / excess sludge = 3.3
As per value furnished in Appendix Appendix A 5.12.

89. Return / excess sludge MLSS concentration = concentration factor x MLSS
= 3.3 x 4500 = 14850 mg/l

90. Cells in aeration tank = MLSS x Volume of aeration tank / 1000
= 4500 x 2000 / 1000 = 9000 Kg

91. Cells wasted per day from the system = Excess sludge = 202.4913 Kg/day

92. Volume of Excess sludge = 202.4913 x 10^6/(14850 x1000) = 13.6358 m^3/day

93. Resulting SRT = θ_c = Cells in aeration tank / Cells wasted per day
= 9000 / 202.4913 = 44.4463 days.

94. Least SRT (Design, Resulting) = Least (25, 44.4463) = 25 days.

95. Volume of excess sludge for least SRT = 13.6358 x 44.4463 / 25 = 24.2424 m^3/day

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96. Excess sludge pump set duty = 24.2424 m^3/day

97. Recirculation ratio = 0.9

AS per Table 5.9 of CPHEEO manual, recirculation ratio varies between 0.25 & 0.8. The maximum value is considered in design.

98. Return sludge pump set duty as MLD = 0.9 x 2 = 1.8 MLD.

Similar design has been carried out for 1 MLD and 5 MLD reactors. The summary of the designs is presented in Table 2.29. Capacity of the blower is obtained from the aeration required based upon the manufacturer's specfications

Table 2. 249 Summary of design of extended aeration reactor

Component	**Capacity**		
	1 MLD`	**2 MLD**	**5 MLD**
EA Reactor details			
No. of aeration tanks	1	1	1
L (m)	13.5	19.25	30.25
B (m)	13.5	19.25	30.25
D (m)	6.00	6.00	6.00
Aeration required			
m^3/hour	1609.6140 m^3/hour	3219.2280 m^3/hour	8048.0701 m^3/hour

2.10 Design of Moving Bed Biofilm Reactor (MBBR)

The moving Bed Biofilm Reactor (MBBR) is a unique modification of the activated sludge process which combines suspended growth and attached growth in the same reactor. The aeration tank of MBBR process carries suspended media which acts as carriers for attached growth of biofilms. The movement of the biofilm carrier is achieved by agitation of air bubbles. This compact system is effective in removal of BOD as well as nutrients.

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The technology was introduced in the 1980s and has since then gained good acceptance for treating municipal sewage as well as industrial wastewater. The treatment efficiency is increased considerably by introducing plastic carriers having a very large internal surface area as suspended media. The surface area in the carriers optimizes the contact of water, air, and the bacteria.

The MBBR technology is compact and hence has a smaller foot print. It is easy to maintain and work well for a high volume of load. By varying the percentage filling of carriers in the tank, the treatment capacity can be easily varied. Perhaps the most significant difference with other activated sludge processes is the elimination of return activated sludge.

2.10.1 Working of MBBR

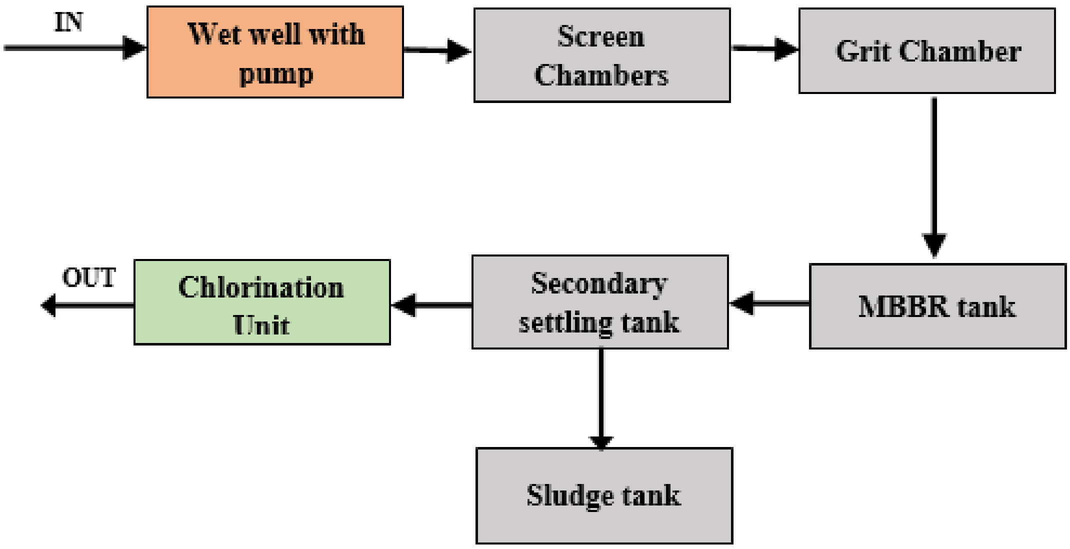

Figure 2.16 Process flow diagram of MBBR

The aeration tank of MBBR is similar to an activated sludge tank. Aeration is done by installing a grid of diffusers in the bottom of the tank which keeps in motion the activated sludge and carriers. The suspended carriers provide large surface area for biofilm growth. Carriers are made of materials with a density close to that of water (HDPE carriers with specific gravity of 0.9. Air bubbles help in moving the carriers thereby ensuring good contact between the wastewater and the biomass on the carriers. Figure 2.16 illustrates the flow diagram of MBBR process.

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MBBR carriers are cylinders having a cross and fins to increase the surface area. The standard used is below 70% of carriers in an area of not more than 465 m^2 per m^3. In most of the cases, two aeration tanks are provided in MBBR technology. The first tank is the reaction tank and the second tank is the stabilization tank.

The key design parameter for sizing the MBBR tank is the surface area loading rate (SALR). Using design values for wastewater flow rate and BOD concentration entering the MBBR tank, the loading rate in g BOD/day can be calculated. Then dividing BOD loading rate in g/day by the SALR in g/m^2/day gives the required carrier surface area in m^2. The organic loading is taken as 1.2 Kg/m^3/day. The carrier filling factor is taken as 50%.

2.10.2 Design Calculations of MBBR

In the present study a single stage process is employed for reducing the BOD from 250 to 30 mg/l. The design calculations are presented in Table 2.30.

Table 2. 30 Design calculations of 1, 2 and 5 MLD MBBR reactors

DESIGN OF 1 MLD, 2 MLD AND 5 MLD MBBR BASED STP

S. No.	Item	1 MLD		2 MLD		5 MLD	
		Value	Unit	Value	Unit	Value	Unit
	INPUT DATA AND SEWAGE CHARACTERISTICS						
1	Inflow rate into the STP	1.00	MLD	2.00	MLD	5.00	MLD
2	Inflow rate into the STP	1000.00	m^3/day	2000.00	m^3/day	5000.00	m^3/day
3	Inflow rate into the STP	0.01	m^3/s	0.02	m^3/s	0.06	m^3/s
4	Peak factor	3.00	Unitless	3.00	Unitless	3.00	Unitless
5	Peak flow into the STP	3.00	MLD	6.00	MLD	15.00	MLD
6	Peak flow into the STP	3000.00	m^3/day	6000.00	m^3/day	15000.00	m^3/day

7	Peak flow into the STP	0.03	m^3/s		0.07	m^3/s		0.17	m^3/s
8	Temperature	14.50	ºC		14.50	ºC		14.50	ºC
9	Influent BOD	250.00	mg/l		250.00	mg/l		250.00	mg/l
	MBBR DESIGN PARAMETERS								
10	BOD surface area loading rate (SALR)	7.5	$g/m^2/day$		7.5	$g/m^2/day$		7.5	$g/m^2/day$
11	BOD volumetric loading rate	1.2	$Kg/m^3.day$		1.2	$Kg/m^3.day$		1.2	$Kg/m^3.day$
12	SARR/SALR	0.9			0.9			0.9	
13	No. of tanks	1.0	No.		1	No.		1	No.
14	Carrier specific surface area	396.0	m^2/m^3		396	m^2/m^3		396	m^2/m^3
15	Carrier specific weight	20.0	Kg/m^3		20	Kg/m^3		20	Kg/m^3
16	Design carrier fill	40%			40%			40%	
17	Carrier void space	60%			60%			60%	
	EFFLUENT STANDARDS								
18	Effluent TSS	100.00	mg/l		100.00	mg/l		100.00	mg/l
19	Effluent BOD	30.00	mg/l		30.00	mg/l		30.00	mg/l
20	Effluent COD	250.00	mg/l		250.00	mg/l		250.00	mg/l
	DETERMINATION OF REACTOR SIZE								
	Size based on Volumetric Loading								
21	BOD load	250000	g/day		500000	g/day		1250000	g/day
22	BOD load	250	Kg/day		500	Kg/day		1250	Kg/day

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23	Volume of tank	208.33	m^3		416.67	m^3		1041.67	m^3
	Size based on SALR								
24	BOD load	250000	g/day		500000	g/day		1250000	g/day
25	Carrier surface area required	33333.33	m^2		66666.67	m^2		166666.67	m^2
26	Calculated Carrier volume required	84.18	m^3		168.35	m^3		420.88	m^3
27	Tank liquid volume required	210.44	m^3		420.88	m^3		1052.19	m^3
28	Tank volume required	176.77	m^3		353.54	m^3		883.84	m^3
29	Chosen tank volume Maximum of (23) and (28)				416.67	m^3			
30	Adopted volume of the tank	215	m^3		425	m^3		1055	m^3
31	Depth of liquid in the tank	4	m		4	m		4	m
32	Diameter of the tank	8.19	m		11.58	m		18.31	m
33	Adopted diameter of the tank	8.25	m		11.75	m		18.5	m
	CHECKING OF DESIGN PARAMETERS								
34	Check HRT for design average flow	254.55	Minutes		254.55	Minutes		254.55	Minutes
35	Check HRT for peak hourly flow	84.85	Minutes		84.85	Minutes		84.85	Minutes
36	BOD surface area removal rate, SARR	6.75	$g/m^2/day$		6.75	$g/m^2/day$		6.75	$g/m^2/day$
37	Estimated BOD removal rate	225000	g/day		450000	g/day		1125000	g/day
38	Effluent BOD concentration	25	mg/l		25	mg/l		25	mg/l
	AIR REQUIREMENT BASED ON RULE OF THUMB								

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	Standard data for calculations							
39	Oxygen needed per Kg BOD	1.5	Kg O_2/Kg BOD		1.5	Kg O_2/Kg BOD		1.5	Kg O_2/Kg BOD
40	SOTE as a function of depth	2.50%	% per m depth		2.50%	% per m depth		2.50%	% per m depth
41	AOTE / SOTE	0.5			0.5			0.5	
42	Pressure drop across diffuser	0.03	bar		0.03	bar		0.03	bar
43	Depth of diffusers	3.7	m		3.7	m		3.7	m
44	Standard Temperature	20	°C		20	°C		20	°C
45	Standard Pressure	1.01	bar		1.01	bar		1.01	bar
46	Atmospheric pressure	1.01	bar		1.01	bar		1.01	bar
47	Air density at STP	1.2	Kg/m^3		1.2	Kg/m^3		1.2	Kg/m^3
48	Oxygen content in air	0.28	Kg/m^3		0.28	Kg/m^3		0.28	Kg/m^3
	CALCULATION OF OXYGEN REQUIREMENT								
49	Oxygen required	337.50	Kg/day		675.00	Kg/day		1687.5 0	Kg/day
50	SOTE	0.09			0.09			0.09	
51	AOTE	0.05			0.05			0.05	
52	Air requirement	18.29	SCMM		36.59	SCCM		91.47	SCMM
53	Blower Outlet Pressure	1.41	bar absolute		1.41	bar absolute		1.41	bar absolute

Green cells: Input variable data to be provided Yellow cells: standard data from CPHEEO manual or other standards Red Cells: Design output

2.10.2.1 Design Steps of 2 MLD Moving Bed Biofilm Reactor (MBBR)

REFERENCES FOLLOWED

DRAFT FOR DISCUSSION PURPOSES ONLY SHOULD NOT BE PRINTED OR REPRODUCED

A. Manual on Sewerage and Sewage Treatment Systems, CPHEEO, November 2013.

B. Waste Water Engineering Treatment and Reuse, Metcalf and Eddy, 5th edition, 2014, McGraw Hill education.

C. Biological Wastewater Treatment Processes II: MBBR Processes, Harlan H. Bengtson, Continuing Education and Development, Inc. 22 Stonewall Court Woodcliff Lake, NJ 07677

Input Data of Sewage Characteristics

1. Influent flow rate = Q_o = 2 MLD

   The design is being undertaken for a 2 MLD STP based on SBR technology.

2. Influent flow rate = 2000 m^3/day

   2 MLD = 2 x 10^6 / 10^3 = 2000 m^3/day

3. Influent flow rate = 2000 / (24 x 60 x 60) = 0.02315 m^3/s

4. Peak factor = 3

5. Peak influent flow rate = 2 x 3 = 6 MLD

6. Peak influent flow rate = 6 x 10^6 / 10^3 = 6000 m^3/day

7. Peak influent flow rate = 6000 / (24 x 60 x 60) = 0.06944 m^3/s

8. Temperature = 14.5°C

   In the present study, the plant is assumed to be located at a latitude of 21.14o N (Central India) where the lowest winter temperature is obtained as 14.5 C from the IMD records.

   T = 14.5° C

9. Influent BOD_5 = 250 mg/l

   As per Table 5.4 of the CPHEEO manual:

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Per capital contribution of BOD_5 = 27.0 g /c / day

With 135 lpcd water supply and sewage generation at 80% of water supplied, the BOD_5 concentration is obtained as:

BOD_5 = 27 x 1000 / (135 x 0.8) = 250 mg/l.

10. BOD Surface area loading rate (SALR) = 7.5 g / m^2 / day

The SALR is obtained from Table 9.15 of Metcalf and Eddy. The given range is 5-15 g / m^2.day.

Professor Hallvard Odegaard of Norway, known as the 'Father of MBBR' has presented the following table in 'The Moving Bed Biofilm Reactor', H. Odegaard, Igarashi, T. et. al., Hokkaido Press, 1999. From the Table it is found that for 90-95% BOD removal the recommended design loading rate is 7.5 g / m^2 / day.

Table 4 from 'The Moving Bed Biofilm Reactor by H. Odegaard

Table 4. Typical design values for KMT reactors at 15 °C

Purpose	Treatment ambition % removal	Design loading rate, g/m^{2}d	Design loading rate kg/m^{3}d at 67 % fill
BOD-removal			
High-rate	75-80 (BOD_7)	25 (BOD_7)	8 (BOD_7)
Normal rate	85-90 (BOD_7)	15 (BOD_7)	5 (BOD_7)
Low rate	90-95 (BOD_7)	7,5 (BOD_7)	2,5 (BOD_7)

11. BOD volumetric Loading rate = 1.2 Kg / m^3.day

The BOD volumetric loading of 1.2 Kg / m^3.day is obtained from clause 5.18.13.5 of CPHEEO manual wherein the specified range is 1.0 to 1.4 Kg / m^3.day

12. SARR/SALR ratio = 0.9 for SALR of 7.5 Kg / m^3.day

As per reference C, 'Use of an estimated surface area removal rate (SARR) allows calculation of the estimated effluent concentration of the parameter being removed. That is, for BOD removal, the estimated effluent BOD concentration can be calculated. Based on graphs and tables provided in several reference, the SARR/SALR is found to range from about 0.8 to nearly 1.0 over the range of SALR values typically used. The SARR/SALR ratio is nearly one at very low SALR values and decreases as the SALR value increases.

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It is to be noted that the ratio SARR/SALR is equal to the % BOD removal expressed as a fraction. This following table is extracted from Reference 'C'.

Table from Biological Wastewater Treatment Processes II: MBBR Processes by Harlan H. Bengtson

Typical Design Values for MBBR reactors at 15°C		
Purpose	Treatment Target % Removal	Design SALR g/m^2-d
BOD Removal		
High Rate	75 - 80 (BOD_7)	25 (BOD_7)
Normal Rate	85 - 90 (BOD_7)	15 (BOD_7)
Low Rate	90 - 95 (BOD_7)	7.5 (BOD_7)

The SARR/SALR ratio can be taken as the mean of the treatment target for each SALR value given in the table above. The interpolation formula can be constructed based on the values furnished below

S. No.	Mean SARR/SALR	SALR $g/m^2/d$
1	(90+95)/2 = 92.5	7.5
2	(85+90)/2 = 87.5	15.0
3	(75+80)/2 = 77.5	25.0

13. No. of tanks = 1

Most of the MBBR processes employ two tanks in order to achieve the desired level of treatment. Since it is not proposed to include nitrogen (Ammonia) removal in the process, a single stage BOD removal tank is considered.

14. Carrier specific surface area = 396 m^2 / m^3

This input is based on the information provided by the carrier supplier.

15. Carrier specific weight = 20 Kg/m^3

This input is based on the information provided by the carrier supplier.

16. Design carrier fill in the MBBR tank = 40%

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As per CPHEEO manual, the carrier volume in Linpor process is 22% and in Pegasus Bio-cube process is 10-20%.

As per reference 'C' the carrier filling varies between 30% and 70%. The lower percentage is a conservative option as it permits for subsequent augmentation of flow or lowering of the SALR by adding more carrier.

17. Carrier void space = 60%

This input is based on the information provided by the carrier supplier.

Effluent Standards

18. TSS = 100 mg/l

19. BOD_5 = 30 mg/l

20. COD = 250 mg/l

With reference to Table 5.3 of the CPHEEO manual, the permitted level of TSS, COD and BOD_5, for discharge into inland surface water bodies is considered. The table is based on General standards for Discharge of Environmental Pollutants, Part A: Effluents. The present design is executed in order to meet the above mentioned limits.

Determination of Reactor Size

i. Size based on volumetric loading

21. BOD load = BOD_5 x Inflow rate = 250 x 2000 = 500000 g/day

22. BOD load = 500000/1000 = 500 Kg/day

23. Size of the tank = BOD load / BOD volumetric loading rate = 500 / 1.2 = 416.6667 m^3

ii. Size based on SALR

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24. BOD load = 500000 g/day from (21) above

25. Carrier surface area required = BOD load / SALR = 500000 / 7.5 = 66666.6667 m^2

26. Calculated carrier volume required = carrier surface area required / Carrier specific surface area = 66666.6667 / 396 = 168.3502 m^3

27. Tank liquid volume required = Carrier volume required / Design carrier fill = 168.3502 / 0.4 = 420.8754 m^3

28. Tank volume required = Tank liquid volume required – (1 – carrier void space) x calculated carrier volume required = 420.8754 – (1 – 0.6) x 168.3502 = 353.5353 m^3

29. Chosen tank volume = higher of the size based on volumetric loading and SALR = Max[416.6667, 353.5353] = 416.6667 m^3

30. Adopted tank volume = 425 m^3

31. Depth of the tank = 4.0 m (Assumed)
Both CPHEEO manual and Metcalf and Eddy are silent on the depth of the MBBR tank. However, depth of 3 to 7 m has been used in design examples.

32. Diameter of the tank = $\sqrt{4(\frac{Volume}{depth})/\pi} = \sqrt{4(\frac{420.8754}{4.0})/\pi} = 11.5774\ m$

33. Adopted diameter of the tank = 11.75 m

Checking of Design Parameters

34. Hydraulic Retention Time (HRT) based on design average flow = Tank volume / Inflow = 353.5353/ 2000 / (24 x 60) = 254.5454 minutes = 4.2424 hours

35. HRT based on peak hourly flow = HRT based on average flow / peak factor = 254.5454 / 3 = 84.8485 minutes = 1.4141 hours.

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36. BOD surface area removal rate, SARR = SALR x Percentage BOD removal
= 7.5 x 0.9 = 6.75 g / m^2 / day

37. Estimated BOD removal rate = SARR x carrier surface area
= 6.75 x 66666.6667 = 450000 g / day

38. Effluent BOD concentration = (BOD Load – BOD removal rate) / Inflow
= (500000 – 450000) / 2000 = 25 mg / l

Air Requirement based on Rule of Thumb

Standard data for calculations

39. Oxygen needed per Kg BOD = 1.5 Kg O_2 / Kg BOD
The rule of thumb for BOD digestion is obtained from Reference 'C', which is stated as, 'For biological treatment with SRT from 5 to 10 days, Kg oxygen required /Kg BOD removed is typically in the range from 0.92 - 1.1 Kg O_2 / Kg BOD. Higher SRT results in a higher value of Kg O_2 required / Kg BOD removed. For a typical SRT, in the MBBR process, this value would be about 1.5 Kg O_2 / Kg BOD removed.

40. SOTE as a function of depth = 2.5% per m depth

41. AOTE / SOTE = 0.5

42. Pressure drop across diffuser = 0.03 bar
The data for items 40, 41 and 42 is obtained from the supplier of diffusers.

43. Depth of diffusers = 4.0 – 0.3 = 3.7 m.
It is assumed that the diffusers are placed 30 cm above the tank bed.

44. Standard temperature = 20^o C

45. Standard Pressure = 1.014 bar

46. Atmospheric Pressure = 1.014 bar

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47. Air density at STP = 1.2 Kg / m^2

48. Oxygen content in the air = 0.277 Kg / m^3

The data for items 44 to 48 is standard information available in any scientific data book.

Calculations for oxygen requirement

49. Oxygen required = Oxygen / Kg BOD x Kg BOD removal

= 1.5 x 450000/1000 = 675 Kg/day

50. SOTE = SOTE as a function of depth x depth of diffusers = (2.5/100) x 3.7 = 0.0925

51. AOTE = SOTE x AOTE/SOTE ratio = 0.0925 x 0.5 = 0.04625

52. Air requirement = $\frac{\frac{Oxygen\ required}{AOTE}}{Oxygen\ content\ in\ ai} = \frac{\frac{675}{0.04625}}{0.277(24)(60)} =$ 36.5889 SCMM

53. Blower outlet pressure = $P_{atm} + P_{drop\text{-}diffuser} + \gamma_{water}$ x diffuser depth

= 1.014 + 0.03 + 9810 x 3.7 / 10^5 = 1.4070 bar absolute

The design summary of 1, 2 and 5 MLD MBBR reactors is given in Table 2.31.

Table 2. 31 Summary of design of 1, 2 and 5 MLD MBBR reactors

Component	**Capacity**		
	1 MLD`	**2 MLD**	**5 MLD**
SBR Reactor details			
No. of aeration tanks	1	1	1
Diameter (m)	8.25	11.75	18.50

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D (m)	4.5	4.5	4.5
Aeration required			
Standard cubic meter per minute (SCMM)	18.3	36.6	91.5
Pressure (bar absolute)	1.41	1.41	1.41

2.11 Design of Trickling Filter (Bio-Tower)

A Bio-Tower is basically a modernized trickling filter which is designed as a high rate or roughening tower for BOD reduction in which wastewater trickles through a bed of PVC structured media which provides larger surface area to the tune of 100 m^2/m^3. The treatment is by the action of microorganisms in the biological layer formed over the fixed media, utilizing the contents of the wastewater as a food source. From time to time dislodging or washing off of slime from the trickling filter media occurs, something commonly referred to as sloughing, and it is removed in the secondary clarifier.

The common advantages of packed bed bio-tower are as follows:

- Fixed film media provides additional surface area for bio-film to grow on it and degrade the organic impurities that are resistant to biodegradation or may even be toxic to some extent.

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- The overall foot-print for a fixed film process-based system is smaller than the activated sludge process system.
- Due to less sludge wastage, the sludge handling and dewatering facility is smaller compared to the activated sludge process.
- The overall efficiency of two stage trickling filter system or a bio-tower is comparable to any other STP technology.
- The traditional rock pebbles are now replaced with plastic elements

2.11.1. Working of Bio-Tower

Trickling filters provide aerobic treatment of wastewater. Wastewater is generally pumped from the primary sedimentation tank, dispersed over a media bed, and allowed to drain back into the recirculation tank / secondary clarifier. The wastewater may be aerated as it flows through the media bed. A trickling filter uses filtration, adsorption, and assimilation for removal of contaminants from wastewater. Wastewater should flow in a thin film over the media to allow time for treatment.

The treatment scheme employs:

i. Primary sedimentation tank
ii. Trickling filter or bio-tower
iii. Secondary sedimentation tank

The sedimentation tanks are used to separate the suspended solids, which can settle by gravity when the sewage is held in a tank. The primary clarifier is located after screens and grit chambers and reduces the organic load on trickling filter. 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 trickling filter and is used to separate the bio-flocculated solids or bio flocs 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.

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2.11.2. Design calculations of Trickling Filter (Bio-Tower)

The design calculations of 1 MLD, 2 MLD and 5 MLD Bio-Tower are presented in Table 6.21. The design is based on 'Wastewater Engineering, Treatment and Reuse' by Metcalf and Eddy, 4th edition, 2003. The design parameters are also taken from the same reference book. Relevant requirements of CPHEEO manual are also adopted. The summary of the design outcome for 1 MLD, 2 MLD and 5 MLD Bio Tower plants is presented in Table 2.32. The design summary of three capacities of the Bio Tower Plant is presented in Table 2.33.

Table 2. 32 Design calculations of 1 MLD, 2 MLD and 5 MLD Bio-Tower

DESIGN OF TRICKLING FILTER (BIO-TOWER) OF CAPACITY 1, 2 AND 5 MLD

S. No.	Item	1 MLD		2 MLD		5 MLD	
		Value	Unit	Value	Unit	Value	Unit
	INPUT DATA AND SEWAGE CHARACTERISTICS						
1	Inflow rate into the STP	1.00	MLD	2.00	MLD	5.00	MLD
2	Inflow rate into the STP	1000.00	m^3/day	2000.00	m^3/day	5000.00	m^3/day
3	Inflow rate into the STP	0.0116	m^3/s	0.0231	m^3/s	0.0579	m^3/s
4	Peak factor	3.00		3.00		3.00	
5	Peak flow into the STP	3.00	MLD	6.00	MLD	15.00	MLD
6	Peak flow into the STP	3000.00	m^3/day	6000.00	m^3/day	15000.00	m^3/day
7	Peak flow into the STP	0.0347	m^3/s	0.0694	m^3/s	0.1736	m^3/s
	INFLUENT (RAW SEWAGE) CHARACTERISTICS						
8	BOD_5	250.00	mg/l	250.00	mg/l	250.00	mg/l
9	COD	425.00	mg/l	425.00	mg/l	425.00	mg/l
10	TSS	375.00	mg/l	375.00	mg/l	375.00	mg/l

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11	Temperature	14.50	ºC	14.50	ºC	14.50	ºC
	EFFLUENT (TREATED SEWAGE) CHARACTERISTICS						
12	BOD	30.00	mg/l	30.00	mg/l	30.00	mg/l
13	COD	250.00	mg/l	250.00	mg/l	250.00	mg/l
14	TSS	100.00	mg/l	100.00	mg/l	100.00	mg/l
	DESIGN PARAMETERS AND ASSUMPTIONS						
15	Number of towers	1.00	No.	1.00	No.	1.00	No.
16	Standard Temperature	20.00	ºC	20.00	ºC	20.00	ºC
17	Hydraulic loading	10 to 75	m^3/m^2.day	10 to 75	m^3/m^2.day	10 to 75	m^3/m^2.day
18	Organic loading	0.6 to 3.2	Kg BOD/m^3.day	0.6 to 3.2	Kg BOD/m^3.day	0.6 to 3.2	Kg BOD/m^3.day
19	Recirculation ratio	1 to 2		1 to 2		1 to 2	
	DESIGN OF PRIMARY SETTLING TANK WITH SLUDGE RETURN						
20	Overflow rate (Average flow) with excess sludge return	25.00	m^3/m^2/day	25.00	m^3/m^2/day	25.00	m^3/m^2/day
21	Overflow rate (Peak flow) with excess sludge return	50.00	Kg/m^2/day	50.00	Kg/m^2/day	50.00	Kg/m^2/day
22	Side water depth	2.50	m	2.50	m	2.50	m
23	Weir loading	125.00	m^3/m/day	125.00	m^3/m/day	125.00	m^3/m/day
24	BOD removal	30.00	%	30.00	%	30.00	%
25	SS removal	60.00	%	60.00	%	60.00	%
26	No. of primary settling tanks	1.00	No.	1.00	No.	1.00	No.
27	Surface area of each settling tank based on average flow	40.00	m^2	80.00	m^2	200.00	m^2
28	Surface area of each settling tank based on peak flow	60.00	m^2	120.00	m^2	300.00	m^2

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29	Chosen area (Max of average and peak)	60.00	m^2	120.00	m^2	300.00	m^2
30	Diameter of each settling tank	8.74	m	12.36	m	19.55	m
31	Adopted diameter of settling tank	8.75	m	12.50	m	19.50	m
32	Total depth of each tank	3.25	m	3.25	m	3.25	m
33	Volume of each tank	150.00	m^3	300.00	m^3	750.00	m^3
34	Weir loading rate	36.43	m^3/m/day	51.52	m^3/m/day	81.45	m^3/m/day
35	BOD of the raw sewage from the settling tank, S2	175.00	mg/l	175.00	mg/l	175.00	mg/l
	DESIGN OF BIO-TOWER - SIZE						
36	Average flow	11.57	l/s	23.15	l/s	57.87	l/s
37	K1	0.21		0.21		0.21	
38	Standard packing depth, D1	6.10	m	6.10	m	6.10	m
39	Standard BOD, S1	150.00	mg/l	150.00	mg/l	150.00	mg/l
40	Site specific packing depth, D2	6.00	m	6.00	m	6.00	m
41	Normalized value of K = K2 @ 20° C	0.1960	g/m^3	0.1960	g/m^3	0.1960	g/m^3
42	K2 @ site temperature	0.1622	g/m^3	0.1622	g/m^3	0.1622	g/m^3
43	Numerator of Hydraulic Application rate	0.9735		0.9735		0.9735	
44	Denominator of Hydraulic Application rate	1.76		1.76		1.76	
45	Constant characteristic of packing used (n)	0.50		0.50		0.50	
46	Hydraulic application rate, q	0.3047	l/m^2.s	0.3047	l/m^2.s	0.3047	l/m^2.s
47	Hydraulic application rate, q	18.28	l/m^2.min	18.28	l/m^2.min	18.28	l/m^2.min
48	Required filter area	37.99	m^2	75.98	m^2	189.94	m^2
49	Required packing volume	227.93	m^3	455.86	m^3	1139.65	m^3

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50	No. of towers	1.00		1.00		1.00	
51	Area of each tower	37.99	m^2	75.98	m^2	189.94	m^2
52	Diameter of each tower	6.96	m	9.84	m	15.56	m
53	Adopted diameter of each tower	7.00	m	10.00	m	16.00	m
	RECIRCULATION RATE						
54	Minimum wetting rate	0.50	$l/m^2.s$	0.50	$l/m^2.s$	0.50	$l/m^2.s$
55	Recirculation rate, q_r	0.20	$l/m^2.s$	0.20	$l/m^2.s$	0.20	$l/m^2.s$
56	Recirculation ratio	0.39		0.39		0.39	
57	Pumping rate	18.99	m^3/h	37.99	m^3/h	94.97	m^3/h
	FLUSHING AND NORMAL DOSE						
58	BOD loading	0.77	Kg $BOD/m^3.day$	0.77	Kg $BOD/m^3.day$	0.77	Kg $BOD/m^3.day$
59	Flushing dose rate	300.00	mm / pass	300.00	mm / pass	300.00	mm / pass
60	Operating dose rate	46.00	mm / pass	46.00	mm / pass	46.00	mm / pass
	DISTRIBUTOR SPEED						
61	Hydraulic application rate, q in $m^3/m^2/hour$	1.10		1.10	$m^3/m^2/hour$	1.10	$m^3/m^2/hour$
62	Distributor speed for flusing	0.0424	RMP	0.0424	RPM	0.0424	RPM
63	Distributor speed for operation	0.2763	RMP	0.2763	RPM	0.2763	RPM
	DESIGN OF SECONDARY SETTLING TANK						
64	Overflow rate (Average flow)	15.00	$m^3/m^2/day$	15.00	$m^3/m^2/day$	15.00	$m^3/m^2/day$
65	Overflow rate (Peak flow)	40.00	$Kg/m^2/day$	40.00	$Kg/m^2/day$	40.00	$Kg/m^2/day$
66	Side water depth	3.00	m	3.00	m	3.00	m
67	Weir loading	185.00	$m^3/m/day$	185.00	$m^3/m/day$	185.00	$m^3/m/day$
68	No. of secondary settling tanks	1.00	No.	1.00	No.	1.00	No.
69	Surface area of each settling tank for average flow based on overflow rate	66.67	m^2	133.33	m^2	333.33	m^2
70	Surface area of each settling tank for peak	75.00	m^2	150.00	m^2	375.00	m^2

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	flow based on overflow rate						
71	Chosen area (Max of average and peak)	75.00	m^2	150.00	m^2	375.00	m^2
72	Diameter of each settling tank	9.77	m	13.82	m	21.86	m
73	Adopted diameter of settling tank	10.00	m	14.00	m	22.00	m
74	Total depth of each tank	3.75	m	3.75	m	3.75	m
75	Volume of each tank	225.00	m^3	450.00	m^3	1125.00	m^3
76	Weir loading rate	32.58	m^3/m/day	46.08	m^3/m/day	72.86	m^3/m/day

Green cells: Input variable data to be provided Yellow cells: standard data from CPHEEO manual or other standards Red Cells: Design output

2.11.2.1 Design Steps of 2 MLD Trickling Filter (Bio-Tower)

REFERENCES FOLLOWED

A. Manual on Sewerage and Sewage Treatment Systems, CPHEEO, November 2013.

B. Waste Water Engineering Treatment and Reuse, Metcalf and Eddy, 4th edition, Tata McGraw Hill edition.

Input Data of Sewage Characteristics

1. Influent flow rate = 2 MLD

   The design is being undertaken for a 2 MLD STP based on Bio-tower technology.

2. Influent flow rate = 2000 m^3/day

   2 MLD = 2 x 10^6 / 10^3 = 2000 m^3/day

3. Influent flow rate = 2000 / (24 x 60 x 60) = 0.023148 m^3/s

4. Peak factor = 3

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5. Peak influent flow rate = 2 x 3 = 6 MLD

6. Peak influent flow rate = 6 x 10^6 / 10^3 = 6000 m^3 /day

7. Peak influent flow rate = 6000 / (24 x 60 x 60) = 0.0694 m^3 /s

Raw Sewage Characteristics for Design

8. Influent BOD_5 = 250 mg/l
As per Table 5.4 of the CPHEEO manual:
Per capital contribution of BOD_5 = 27.0 g /c / day
With 135 lpcd water supply and sewage generation at 80% of water supplied, the BOD_5 concentration is obtained as:
BOD_5 = 27 x 1000 / (135 x 0.8) = 250 mg/l.

9. COD of influent = 425 mg/l
The value of COD of influent sewage is taken from Table 5.4 of CPHEEO manual.

10. Influent Total Suspended Solids (TSS) = 375 mg/l
The value of TSS of influent sewage is taken from Table 5.4 of CPHEEO manual.

11. Temperature = 14.5^{o}C
In the present study, the plant is assumed to be located at a latitude of 21.14o N (Central India) where the lowest winter temperature is obtained as 14.5 C from the IMD records.
T = 14.5^o C

Treated Effluent Standards

12. BOD = 30 mg/l

13. COD = 250 mg/l

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14. TSS = 100 mg/l

With reference to Table 5.3 of the CPHEEO manual, the permitted level of BOD_5, COD, TSS and NH_4-N for discharge into inland surface water bodies is considered. The table is based on General standards for Discharge of Environmental Pollutants, Part A: Effluents. The present design is executed in order to meet the above-mentioned limits.

Design Parameters and Assumptions

15. Number of towers = 2
In order to ensure continuous operation of the plant, a minimum of 2 tanks may be provided so that that one tank is functioning when the other is under maintenance.

16. Standard (Reference) temperature = 20°C.
Used in equation 9-16 of Metcalf and Eddy for temperature correction.

17. Hydraulic loading = 10 to 75 m^3/m^2

18. Organic loading = 0.6 to 3.2 Kg BOD / m

19. Recirculation ratio = 1 to 2

The values of (17), (18) and (19) have been taken from Table 9.1 of Metcalf and Eddy.

Design of Primary Settling Tank with Sludge Return

20. Overflow rate for average flow with excess sludge return = 25 m^3/m^2/day
The overflow rate is obtained from Table 5.8 of CPHEEO manual. The smaller value is taken as the plant capacity is less than 5 MLD.

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21. Overflow rate for peak flow with excess sludge return = 50 m^3/m^2/day
The overflow rate is obtained from Table 5.8 of CPHEEO manual. The smaller value is taken as the plant capacity is less than 5 MLD.

22. Side water depth = 2.5 m
The side water depth is obtained from Table 5.8 of CPHEEO manual.

23. Weir loading rate for settling with excess sludge return = 125 m^3 / m / day

24. BOD removal in primary settling tank = 30%
The percentage BOD removal in primary settling tank is based on section 5.7.5 of CPHEEO manual, wherein it is mentioned, '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.'

25. Suspended Solids removal in primary settling tank = 60%.
The percentage removal is based on section 5.7.5 of CPHEEO manual as stated above.

26. No. of primary settling tanks = 1
Usually two tanks are provided in order to facilitate ease of maintenance and repairs.

27. Required surface area of each settling tank based on average flow
= average flow / average flow overflow rate = (2000/1) / 25 = 80 m^2

28. Required surface area of each settling tank based on peak flow
= peak flow / peak flow overflow rate = (6000/1) / 50 = 120 m^2

29. Chosen area of the primary settling tank = Maximum [80, 120] = 120 m^2
Refer to section 5.7.4.2.5 of CPHEEO manual for area of the clarifier

30. Diameter of each settling tank = $\sqrt{\frac{4\,(120)}{\pi}} = 12.36\ m$

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31. Adopted diameter of the settling tank = 12.5 m

32. Total depth of each tank = Assigned depth + free board + sludge zone
=2.5 + 0.5 + 0.25 = 3.25 m

33. Volume of each tank = Area x depth = 120 x 2.5 = 300 m^3

34. Weir loading rate = Flow in each tank / perimeter of the tank
= (2000/1)/(π x 12.36) = 51.52 m^3/m^2/day
This is less than 125 m^3/m^2/day

35. BOD of effluent from the primary settling tank = Influent BOD (1-%removal)
= 250 x (1-0.3) = 175 mg/l

Design of Trickling Filter (Bio-Tower)

36. Average flow = 2 x 10^6/(24 x 60 x 60) = 23.1481 l/s

37. k1 = 0.21
k1 = normalized value of ‘k’ at a depth of 6.1 m and influent BOD of 150 mg/l as obtained from Table 9-6 of Metcalf and Eddy for Domestic wastewater.

38. Standard packing depth D1 = 6.1 m
The standard packing depth corresponds to the depth at which experiments were conducted in order to obtain the value of ‘k’ in equation 9-19 of Metcalf and Eddy.

39. Standard BOD S1 = 150 mg/l
S1 is the influent BOD at which experiments were conducted in order to obtain the value of ‘k’ in equation 9-19 of Metcalf and Eddy.

40. D2 = Packing depth of the trickling filter (Bio-tower) under design = 6 m

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41. k2 = Normalized value of k for site-specific packing depth and influent BOD @ 20^{o}C

The value of k2 is obtained from equation 9-22 of Metcalf and Eddy.

$k_2 = k_1 \left(\frac{D_1}{D_2}\right)^{0.5} \left(\frac{S_1}{S_2}\right)^{0.5}$ Equation 9-22 of Metcalf and Eddy

Where k_2 = normalized value of k for the site specific packing depth (D_2) and influent BOD concentration (S_1).

k_1 = k value at depth of 6.1 m and influent BOD of 150 mg/l

S_1 = 150 mg/l BOD

S_2 = Site-specific influent BOD concentration, mg/l

D_1 = 6.1 m packing depth, m

D_2 = Site-specific packing depth, m

$$k_2 = 0.21 \left(\frac{6.1}{6}\right)^{0.5} \left(\frac{150}{175}\right)^{0.5} = 0.1960$$

42. k_2 @ site temperature = 0.1257

From equation 9-20 of Metcalf and Eddy

$K_T = k_{20}\,(1.035)^{T-20}$

T = 14.5^{o}C

$K_{14.5^oC} = 0.1960 \times (1.035)^{(14.5-20)} = 0.1622$

43. Numerator of Hydraulic Application rate equation = 0.9375

The hydraulic application rate is given by equation 9-19 of Metcalf and Eddy.

$\frac{S_e}{S_o} = e^{\frac{-kD}{q^n}}$ Equation 9-19 of Metcalf and Eddy

$$q = \left(\frac{kD}{ln\frac{S_o}{S_e}}\right)^{\frac{1}{n}}$$

where

Se = BOD concentration of settled filter effluent, mg/l

S_o = influent Bod concentration to the filter, mg/l

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k = waste water treatability and packing coefficient based on n = 0.5, $(l/s)^{0.5}/m^2$

D = Packing depth, m

Q = hydraulic application rate of primary effluent, excluding recirculation, l / m^2.s

n = constant characteristic of packing used

Numerator of the hydraulic application rate equation = kD = 0.1622 x 6 = 0.9735

44. Denominator of Hydraulic Application rate equation = 1.7636

$$ln\frac{S_o}{S_e} = ln\frac{175}{30} = 1.7636$$

45. n = constant, characteristic of packing used = 0.5

As per section 9.2 of Metcalf and Eddy, the value of n is normally assumed to be 0.5 and pilot-plant or full-scale plant influent and effluent BOD concentration data are used to solve for k.

46. Hydraulic application rate q = 0.3047 l / m^2.s

$$q = \left(\frac{0.9735}{1.7636}\right)^{\frac{1}{0.5}} = 0.3047 \text{ l / m}^2\text{.s}$$

47. Hydraulic application rate q = 0.3047 x 60 = 18.28 l / m^2.min

48. Required filter area = Average flow / q = 23.1481 / 0.3047 = 75.98 m^2

49. Required packing volume = tower volume = area x height = 75.98 x 6 = 455.862 m^3

50. No. of towers = 1

51. Area of each tower = total area / 1 = 75.98 / 1 = 75.98 m^2

52. Diameter of each tower = $\sqrt{\frac{4(75.98)}{3.14}} = 9.84\ m$

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53. Adopted diameter of each tower = 10.00 m

Recirculation and Pumping rate

54. Minimum wetting area = 0.50 l / m^2.s

The minimum hydraulic application rate recommended by Dow Chemical, as documented in Metcalf and Eddy is 0.5 l / m^2.s to provide maximum efficiency. Shallower tower designs require recirculation to provide minimum wetting rates. When above the minimum hydraulic application rate, recirculation was reported to have little benefit.

55. q_r = recirculation rate

$q + q_r$ = 0.5 l / m^2.s

q_r = 0.50 – 0.3074 = 0.1953 l / m^2.s

If q_r is negative, it indicates that there is no need of recirculation. For tall towers, q_r is usually negative and as the tower height is reduced, q_r becomes positive.

56. Recirculation ration = Recirculation rate / minimum wetting area = 0.1953 / 0.5 = 0.3906 l / m^2.s.

57. Total pumping rate = ($q + q_r$) x required filter area = (0.3047 + 0.1953) x 75.98 = 37.99 m^3/hour

Flushing and Normal Dose

58. BOD loading = $Q S_o / V$

= 2000 x 175 / (455.8616 x 1000) = 0.7678 Kg BOD / m^3.day

59. Flushing dose = 300 mm / pass

60. Operating dose = 46 mm / pass

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The dosing rate on a trickling filter is the depth of liquid discharged on top of the packing fo reach pass of the distributor. Results from various investigations have revealed that lower distributor speed results in better filter performance in terms of BOD removal, reduction in fly population, biofilm thickness and odours. The thinner biofilm creates more surface area and results in more aerobic biofilm. A daily intermittent high dose referred to as a flushing dose is used to control the biofilm thickness. The recommended operating and flushing dozing rates as a function of BOD loading is given in Table 9-3 of Metcalf and Eddy.

<table>
<tr><th>BOD loading Kg / m³.day</th><th>Operating Dose mm/pass</th><th>Interpolated operating dose mm/pass</th><th>Flushing dose, mm/pass</th></tr>
<tr><td>0.5</td><td>15-45
Adopted 30</td><td rowspan="2">30 + 0.2678 x 30 / 0.5
= 46</td><td rowspan="2">300</td></tr>
<tr><td>1.0</td><td>30-90
Adopted 60</td></tr>
</table>

61. Convert q from l / m^2 / s to m^3 / m^2 / hour

q = 0.3047 l / m^2 / s = (0.3047 / 1000) x (60 x 60)

q = 1.0969 m^3 / m^2 / hour

62. The rotational speed of the rotary distributor is determined using equation 9-1 of Metcalf and Eddy.

$$n = \frac{(1+R)(q)(1000)}{A\,(DR)(60)}$$

Where

N = rotational speed in revolutions/minute

q = influent applied hydraulic loading rate, m^3 / m^2 / hour

R = recycle ratio = 0 if no recirculation is required

A = number of arms in rotary distributor assembly

DR = dosing rate, mm/pass of distributor arm

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1000 = conversion factor for mm to m

60 = conversion factor for minutes to hour

Distributor speed for flushing is given by:

$$n = \frac{(1 + 0.39)(1.0969)(1000)}{2\ (300)(60)} = 0.0424 \frac{revolution}{minute}$$
$$= 23.61\ minutes/revolution$$

63. Distributor speed for normal operation is given by:

$$n = \frac{(1 + 0.39)(1.0969)(1000)}{2\ (46)(60)} = 0.2763\ \frac{revolution}{minute}$$
$$= 3.6192\ minutes/revolution$$

Design of Secondary Settling Tank

64. Overflow rate for average flow = 15 m^3/m^2/day

The overflow rate is obtained from Table 5.8 of CPHEEO manual. The smaller value is taken as the plant capacity is less than 5 MLD.

65. Overflow rate for peak flow = 40 m^3/m^2/day

The overflow rate is obtained from Table 5.8 of CPHEEO manual. The smaller value is taken as the plant capacity is less than 5 MLD.

66. Side water depth = 3.0 m

The side water depth is obtained from Table 5.8 of CPHEEO manual.

67. Weir loading rate for settling with excess sludge return = 185 m^3 / m / day

The weir loading rate is obtained from Table 5.8 of CPHEEO manual.

68. No. of primary settling tanks = 1

Usually two tanks are provided in order to facilitate ease of maintenance and repairs.

69. Required surface area of each settling tank based on average flow

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= average flow / average flow overflow rate = (2000/1) / 15 = 133.3333 m^2

70. Required surface area of each settling tank based on peak flow

= peak flow / peak flow overflow rate = (6000/1) / 40 = 150 m^2

71. Chosen area of the primary settling tank = Maximum [133.3333, 150] = 150 m^2

Refer to section 5.7.4.2.5 of CPHEEO manual for area of the clarifier

72. Diameter of each settling tank = $\sqrt{\frac{4\,(150)}{\pi}} = 13.8233\ m$

73. Adopted diameter of the settling tank = 14.00 m

74. Total depth of each tank = Assigned depth + free board + sludge zone

=3.0 + 0.5 + 0.25 = 3.75 m

75. Volume of each tank = Area x depth = 150 x 3.0 = 450 m^3

76. Weir loading rate = Flow in each tank / perimeter of the tank

= (2000/1)/(π x 13.8233) = 46.0775 m^3/m^2/day

This is less than 185 m^3/m^2/day

The design outcome is presented in Table 2.32.

Observations:

In the present design, it is noticed that, when the tower height is 10 m the diameter is 7.75 m and the recirculation rate becomes negative, which means there is no need of recirculation. When the tower height is 6 m, the diameter is 10 m and recirculation ratio is 0.39. However, the quantity of packing volume remains same in both the cases.

Table 2. 33 Summary of design of Trickling Filter (Bio-Reactor)

Component	Capacity		
	1 MLD`	2 MLD	5 MLD
Primary Sedimentation Tank			

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No. of tanks	1	1	1
Diameter (m)	8.75	12.50	19.50
D (m)	3.25	3.25	3.25
Bio-Tower			
No. of towers	1	1	1
Diameter (m)	7	10	16
Height (m)	6	6	6
Distributor speed for operation (RPM)	0.2763	0.2763	0.2763
Distributor speed for Flushing (RPM)	0.0424	0.0424	0.0424
Secondary Sedimentation Tank			
No. of tanks	1	1	1
Diameter (m)	8.0	11.50	18.0
D (m)	2.7	2.7	2.7

2.12 Aerated Facultative Ponds

An aerated facultative pond is a variation of the traditional facultative pond except that aerators are employed to augment the oxygen supply from photosynthesis. The aerated facultative pond also utilizes both aerobic and anaerobic proceses in the treatment of waste water. Since photosynthesis is not the primary source of oxygen, the depth of the aerated facultative pond can be more than that of traditional facultative pond since penetration of sunlight is not necessary.

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Aerobic Zone: The system consists of a not so shallow pond with mechanical aeration equipment installed to provide oxygen to the wastewater. The surface layer of the pond is well-aerated due to the installed aeration system. This promotes the growth of aerobic bacteria, which break down organic matter through oxidative processes.

Facultative Zone: The oxygen levels in the pond gradually decrease with depth leading to the formation of a facultative zone near the pond bottom. The anaerobic and facultative bacteria is found to be very active in this zone in the absence of or low concentration of oxygen. These bacteria continue the degradation of organic matter through anaerobic processes.

In this system, wastewater enters the pond from one side and undergoes treatment as it moves from the aerobic surface layer to the deeper facultative zone. Organic matter is degraded by aerobic bacteria at the surface and by facultative bacteria in the deeper layers, leading to the reduction of organic pollutants and pathogens.

2.12.1 Design Calculations of Aerated Facultative Pond after Anaerobic Pond

The design calculations of 1 MLD, 2 MLD and 5 MLD aerated facultative ponds after anaerobic pond are presented in Table 2.34. The design is based on Volume III of Biological Wastewater Treatment Series, Volume Three published by International Water Association (IWA). Relevant requirements of CPHEEO manual are also adopted. The design summary of three capacities is presented in Table 2.35.

It may be noted that design of facultative ponds after anaerobic reactor has been attempted in section 2.4. That design considered three ponds, two parallel ponds primary and one secondary pond in series. The design of an aerated facultative pond is very similar to that of a facultative pond, but with an aerobic pond close to the surface and a deeper anaerobic zone. In the present desing only two parallel ponds are considered on the downstream of the anaerobic pond. As stated above, the depth of the ponds is taken as 3.5 m inclusive of a 1.0 m zone for sludge in both the ponds.

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Table 2. 34 Design calculations of aerated facultative pond after anaerobic pond

DESIGN OF AERATED FACULTATIVE WASTE STABILIZATION POND OF 1, 2 AND 5 MLD CAPACITY

S. No.	Design parameter	Design output					
1	Capacity of the STP	1.00	MLD	2.00	MLD	5.00	MLD
2	Capacity of the STP	1000.00	m^3/day	2000.00	m^3/day	5000.00	m^3/day
3	Capacity of the STP	0.01	m^3/s	0.02	m^3/s	0.06	m^3/s
4	Per capita supply	135.00	LPCD	135.00	LPCD	135.00	LPCD
5	Influent BOD_5 (As per Table 5.4 of CPHEEO Manual - 2013)	27.00	g /capita per day	27.00	g/capita per day	27.00	g/capita per day
6	Influent BOD_5 for Anearobic pond	250.00	mg/l	250.00	mg/l	250.00	mg/l
7	Influent BOD, for Facuiltative pond	127.50	mg/l	127.50	mg/l	127.50	mg/l
8	Effluent BOD_5 (As per Schedule VI of environment (protection) third Amendment Rules, 1993 - for Inland surface water)	30.00	mg/l	30.00	mg/l	30.00	mg/l
9	Average ambient temperature in December	14.50	ºC	14.50	ºC	14.50	ºC
10	Detention time	13.00	days	13.00	days	13.00	days
	Determine effluent soluble BOD_5						
11	BOD removal coefficient K at 20ºC	0.70	day^{-1}	0.70	day^{-1}	0.70	day^{-1}
12	BOD removal coefficient K at design temperature	0.58	day^{-1}	0.58	day^{-1}	0.58	day^{-1}

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13	Soluble BOD_5	14.94	mg/l		14.94	mg/l		14.94	mg/l
	Determine effluent Particulate BOD_5								
14	Suspended Solids in the effluent	50.00	mg/l		50.00	mg/l		50.00	mg/l
15	Particulate BOD_5	15.00	mg/l		15.00	mg/l		15.00	mg/l
	Determine total effluent BOD_5								
16	Total effluent BOD_5	29.94	mg/l		29.94	mg/l		29.94	mg/l
17	Check for the effluent BOD	OK			OK			OK	
18	Efficiency of BOD removal	76.51	%		76.51	%		76.51	%
	Determine dimensions of the aerated facultative pond								
19	Volume required	13000.00	m^3		26000.00	m^3		65000.00	m^3
20	Depth of the pond	3.50	m		3.50	m		3.50	m
21	Pond area required	3714.29	m^2		7428.57	m^2		18571.43	m^2
	Oxygen and Power Requirement								
22	Kg oxygen required to oxidize Kg BOD_5	1.20	KgO_2 / $KgBOD_5$		1.20	KgO_2 / $KgBOD_5$		1.20	KgO_2 / $KgBOD_5$
23	Oxygen required (Standard) per day	117.00	KgO_2/ day		234.00	KgO_2/ day		585.00	KgO_2/ day
24	Oxygen required (Standard) per hour	4.88	KgO_2/ hour		9.75	KgO_2/ hour		24.38	KgO_2/ hour

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25	Oxygenation efficiency in standard conditions (aerator dependent)	1.80	KgO_2/ KWh	1.80	KgO_2/ KWh	1.80	KgO_2/ KWh
26	Field oxygenation efficiency as percentage of standard oxygenration efficiency	60.00	%	60.00	%	60.00	%
27	Oxygenration efficiency in field conditions	1.08	KgO_2 / KWh	1.08	KgO_2 / KWh	1.08	KgO_2 / KWh
28	Tota power required	4.51	KW	9.03	KW	22.57	KW
29	Total power required (say)	5.00	KW	10.00	KW	24.00	KW
30	Power of each aerator	2.50	KW	5.00	KW	12.00	KW
31	No. of aerators to be provided	2.00		2.00		2.00	
32	Average power level in the whole pond	0.38	W/m^3	0.38	W/m^3	0.37	W/m^3
33	No. of ponds in parallel	2.00		2.00		2.00	
34	Population equivalent	8818.34		17636.68		44091.71	
35	Area per person (net)	0.42	m^2	0.42	m^2	0.42	m^2
36	Rate of sludge accumulation	0.07	m^3/ person / year	0.07	m^3/ person / year	0.07	m^3/ person / year
37	Sludge volume in 1 year	617.28	m^3 / year	1234.57	m^3	3086.42	m^3
38	Depth in the cell to accommodate the sludge	1.00	m	1.00	m	1.00	m
39	Volume of sludge in the two cells	3714.29	m^3	7428.57	m^3	18571.43	m^3

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40	Duration of sludge cleaning	6.02	years		6.02	years		6.02	years
41	Area of each pond	1857.14	ha		3714.29	ha		9285.71	ha
42	L/B raio	4.00	Dimen sionless		4.00	Dimen sionless		4.00	Dimen sionless
43	Length L	86.19	m		121.89	m		192.72	m
44	Breadth B	21.55	m		30.47	m		48.18	m
45	Tank L x B X D (Mid-Depth)	86 x 22 x 3.5	m		122 x 30.5 x 3.5	m		193 x 48 x 3.5	m
46	Length at mid-depth of the tanks	86.00	m		122.00	m		193.00	m
47	Width at mid-depth of the tanks	22.00	m		30.50	m		48.00	m
48	Side slope of pond is 1V :2.0 H	2.00	m		2.00	m		2.00	m
49	Top of embankment above mid depth	2.75	m		2.75	m		2.75	m
50	Tank Bottom below mid-depth	1.75	m		1.75	m		1.75	m
51	Top length	97.00	m		133.00	m		204.00	m
52	Top width	33.00	m		41.50	m		59.00	m
53	Bottom length	79.00	m		115.00	m		186.00	m
54	Bottom width	15.00	m		23.50	m		41.00	m

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Green cells: Input variable data to be provided Yellow cells: standard data from CPHEEO manual or other standards Red Cells: Design output

2.12.1.1 Design Steps of 2 MLD Aerobic Facultative Pond after Anaerobic Pond

REFERENCES FOLLOWED

A. Waste Stabilization Ponds, Biological Waste Water Treatment Series, Volume Three, Marcos von Sperling, IWS Publishing, London, 2007.

B. Manual on Sewerage and Sewage Treatment Systems, CPHEEO, November 2013.

C. Wastewater Treatment for Pollution Control and Reuse, third edition, Soli J Arceivala and Shyam R Asolekar, Mc Graw Hill Education, 2007.

Input Data and Sewage Characteristics

1. Capacity of the STP = 2 MLD

   The design is being undertaken for a 2 MLD STP.

2. Capacity of the STP = 2000 m^3 / day

   Convert MLD to m^3 / day = 2 x 10^6 / 1000 = 2000 m^3 / day

3. Capacity of the STP = 0.02315 m^3 / s

   Convert m^3 / day to m^3 / s = 2000 / (24 x 60 x 60) = 2000 m^3 / s

4. Per capita water supply = 135 lpcd

   As per Table 2.1 of CPHEEO's 'Manual on Water Supply and Treatment', the per capital supply for cities provided with piped water supply is 135 lpcd.

5. Influent BOD_5 = 27 g / capita / day

   BOD_5 in the influent municipal sewage = 27 g / capital / day

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As per Table 5.4 of CPHEEO Manual on Sewerage and Sewage Treatment Systems, the per capita contribution of BOD is 27 g /c / day.

6. Influent BOD_5 in the Anaerobic Pond = 250 mg/l

With 135 lpcd water supply and sewage generation at 80% of water supplied, the BOD concentration is obtained as:

BOD_5 = 27 x 1000 / (135 x 0.8) = 250 mg/l.

7. Influent BOD_5 in the facultative pond, S_o = 127.5 mg/l

The facultative pond is employed sequentially after the anaerobic pond. As per the design of anaerobic pond, the BOD removal efficiency is obtained as 49% at a temperature of 14.5^o C.

BOD removal efficiency (in %) = 2T + 20 = 2 x 14.5 + 20 = 49%. The reference for this has been dealt with in the anaerobic pond design section.

Balance BOD after anaerobic pond = 100 -49 = 51%

Influent BOD in the facultative pond = 0.51 x 250 = 127.5 mg/l.

8. Effluent BOD_5 in the treated water = S = 30 mg/l

With reference to Table 5.3 of the CPHEEO manual, the permitted level of BOD_5 for discharge into inland surface water bodies is 30 mg/l. The table is based on 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.

9. Lowest Average ambient temperature = 14.5^{o}C

The lowest average ambient temperature corresponds to the coldest month (December) and the value is obtained from IMD. In the present study the temperature of 14.5^o C is assumed as representational value for latitude of 21.14^o N.

10. Detention time = 13 days.

As per design criteria of Reference A, ‘The detention time should be adopted in orer to allow a satisfactory removal of BOD, according to the theory of kinetics. Usually the

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values adopted vary in the range of 5 to 10 days. In case the required effluent BOD is not attained, the detention time may be suitably increased till the desired effluent quality is attained. In the present study, the time of 13 days is adopted to achieve the target BOD of 30 mg / l.

Determine the effluent soluble BOD_5

11. BOD removal coefficient = 0.7 day^{-1}

As per section 5.8.1.7.12 of Reference B, the overall substrate removal rate 'K' varies from 0.6 to 0.8 per day for soluble BOD at 20^o C.

As per Reference A, the value of BOD removal coefficient K is higher in case of facultative aerated lagoons. Typical values for the complete mix regine are in the range of 0.6 to 0.6 day^{-1}.

The mid value of 0.7 day^{-1} is adopted in the present design.

12. BOD removal coefficient K at design temperature

The K value of 0.7 day^{-1} is valid for a temperature of 20^o C. For other temperatures, as per Reference A, the K value is obtained from equation 2.7 stated as:

$$K_T = K_{20}\ \theta^{(T-20)}$$

Where

K_T = BOD removal coefficient at a temperature of T^o C

K_{20} = BOD removal coefficient at a temperature of 20^o C

T = liquid temperature

Θ = Temperature Coefficient = 1.035

The same equation is presented in Section 5.8.1.7.12 in the form of equation 5.33 in Reference B.

K_{20} = 0.7

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$T = 14.5^o$ C

$\Theta = 1.035$

$K_{14.5} = 0.7 \times 1.035^{(14.5-20)}$

$= 0.5793$

13. Soluble BOD_5 = 14.9450 mg/l

As per Reference A, the solutble BOD_5 is given by the equation

$$S = \frac{S_o}{1 + Kt}$$

S = 127.5 / (1 + 0.5793 x 11) = 14.94 mg / l

The above equation is also given in Reference B as Equation 5.38.

Determine Effluent Particulate BOD_5

14. Suspended solids in the effluent = 50 mg/l

As per Table 5.12 of Reference B, the suspended solids in the aerated lagoon is between 40-150 mg/l.

As per Table 4.1 of Reference A, the suspended solids for a Power level of 0.75 W/m^3 is 50 mg/l. Facultative aerated lagoons work with low power levels in order to facilitate sedimentation of the solids. The value of power level is usually in the range of 0.75 to 1.50 W / m^3.

Hence, based upon the above two references, the suspended solids is taken as 50 mg/l.

15. Particulate effluent BOD_5 = 0.3 to 0.4 mg BOD_5 / mg SS

With reference to section 4.4 of Reference A, each 1 mg/l of suspended solids produce a particulate BOD between 0.3 to 0.4 mg/l.

Particulate BOD_5 = 50 x 0.3 = 15 mg/l

Determine Total Effluent BOD_5

16. Total effluent BOD_5 = Soluble BOD_5 + Particulate (Suspended) BOD_5

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17. Efficiency of BOD removal $= (S_o – S) / S_o$
$= (127.5 – 30) \times 100 / 127.5 = 76.51\%$

Determine Dimensions of Aerated Facultative Pond

18. Volume of the pond = detention time x Q = 13 x 2000 = 26000 m^3
19. Depth of pond = 3.5 m
As per Reference A, the depth of the pond varies between 2.5 to 4.0 m.
As per Table 5.12 of Reference B, the depth varies between 2.5 to 5 m.

20. Pond area required = Volume / depth = 26000 / 3.5 = 7428.57 m^2

Oxygen and Power Requirement

21. Kg oxygen required to oxidize Kg BOD_5 = 1.20 KgO_2 / Kg BOD_5
As per reference A, the Kg O_2 requied to oxidize Kg of carbonaceous BOD_5 ranges between 0.8 to 1.2 Kg.

22. Standard oxygen requirement is given by
O_2 required = a. Q. $(S_o – S)$
Where
a = 1.20 KgO_2 / Kg BOD_5
Q = flow
S_o = influent BOD in mg/l
S = effluent BOD in mg/l
O_2 required = 1.2 x 2000 x (127.5 – 30) / 1000 = 234 Kg O_2 / hour

23. Standard conditions oxygen requirement per hour = 234 / 24 = 9.75 Kg O_2 / hour

24. Oxygenation efficiency of the aerator in standard conditions = 1.8 Kg O_2 / KWh
As per section 5.8.1.7.5.3 of Reference B, the oxygen transfer capacities of surface aerators under standard conditions lies between 1.2 to 2.4 Kg O_2 / KWh

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As per section 4.7 of Reference A, at standard conditions, the oxygenation efficiency of the aerators is within the range of 1.2 to 2.0 Kg O_2 / KWh

25. Oxygenration efficiency of aerators in field conditions = 0.55 to 0.65 times the oxygenraiton efficiency in standard condtions.
    As per section 4.7of Reference A, under real (field) operating conditions in the treatment plant, the oxygenration efficiency is smaller.
    Adopt oxygenration efficiency of aerators in field conditions = 60%
26. Oxygenration efficiency in operating field conditions = 0.6 x 1.8 = 1.08 Kg O_2 / KWh

27. Total power required = 9.75 / 1.08 = 9.0278 KW

28. Total power required (rounded off), say = 10 KW.

29. Power of each aerator = 5 KW

30. Number of aerators to be provided = 10 / 5 = 2 Nos.

31. Average power level in the pond = Power / Volume = 10x1000/26000 = 0.3846 W / m^3

32. No. of ponds in parallel = 2
    Usually two ponds are provided in parallel. This affords flexibility during the occasional periods of sludge removal

33. Population equivalent = 2.00 x 10^6 / (135 x 0.8 x 1.05) = 17637
    Where
    STP capacity = 2 MLD = 2 x 10^6 l / day
    Per capita supply = 135 lpcd
    Sewage generation as a percentage of lpcd = 80%
    Ground water infiltration into sewers = 5%

34. Area of the pond per person = 7428.57 / 17637 = 0.42 m^2 per person

35. Rate of sludge accumulation = 0.05 m^3 / person / year

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As per reference A, the annual accumulation of sludge is recommended as 0.05 m^3 / person / year.

As per section 5.8.1.7.15 sludge accumulation occurs at the rate of 0.03 to 0.05 m^3 per person per year

36. Sludge volume accumulated in 1 year = 0.05 x 17637 = 881.85 m^3

37. Depth provided in the pond to accommodate the sludge = 1.0 m

38. Volume of sludge in the two cells = area of cell the cells x depth of sludge
    = 7428.57 x 1 = 7428.57 m^3

39. Duration of sludge cleaning = 7428.57 / 881.85 = 8.42 years.
    As per section 5.8.1.7.15 of Reference B, the sludge can be cleaned manually once in 5 – 10 years. Hence 8.42 years frequency is satisfactory.

40. Area of each pond = 7428.57 / 2 = 3714.29 m^2

41. L / B ratio adopted for the pond = 4

42. Considering rectangular tank, $L = \sqrt{4A} = \sqrt{4\ x\ 3714.29} = \ 121.89\ m$

43. Breadth of the tank B = 121.89 / 4 = 30.47 m

44. Tank dimensions at mid-depth (rounded off) = 122 x 30.5 x 3.5 m

45. Length at mid-depth of the tank = 122.00 m

46. Width at mid-depth of the tank = 30.5 m

47. Side slope of the pond is 1 V : 2 H = 2.0

48. Top of embankment above mid-depth = 2.75 m
    Assumed free board = 1.0 m

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Top of the embankment above mid-depth = 1.0 + 3.5 / 2 = 2.75 m.

49. Tank bottom below mid- depth = 1.75 m

50. Top length = $L_{Mid\text{-}depth}$ + 2(side slope x top above mid-depth) = 122+2(2x2.75) = 133 m

51. Top Width = $B_{Mid\text{-}depth}$ + 2(side slope x top above mid-depth) = 30.5+2(2x2.75) = 41.50 m

52. Bottom Length = $L_{Mid\text{-}depth}$+2(side slope x top above mid-depth) = 122-2(2x1.75) = 115 m

53. Bottom Width = $B_{Mid\text{-}depth}$ + 2(side slope x top above mid-depth) = 30.5-2(2x1.75) = 23.50 m

Table 2. 35 Summary of 1, 2, and 5 MLD Aerated Facultative Ponds after Anaerobic Pond

Capacity MLD	Component	No. of ponds	Length (m)	Width (B)	Depth (D)
1	Aerated Facultative Pond (Top / Middle / Bottom)	2	97/86/79	33/22/15	3.5
2	Aerated Facultative Pond (Top / Middle / Bottom)	2	133/122/115	41.5/30.5/23.5	3.5
3	Aerated Facultative Pond (Top / Middle / Bottom)	2	204/193/186	59/48/41	3.5

2.12.1.2 Design calculations of Aerated Facultative Pond after UASBR

The design calculations of 1 MLD, 2 MLD and 5 MLD aerated facultative ponds after UASB reactor are presented in Table 2.36. The design is based on Volume III of Biological Wastewater Treatment Series, Volume Three published by International Water Association (IWA). Relevant requirements of CPHEEO manual are also adopted. The design summary of three capacities is presented in Table 2.37.

It may be noted that design of facultative ponds after UASBR has been attempted in section 2.6. That design considered three ponds, two parallel ponds primary and one secondary pond in series. The design of an aerated facultative pond is very similar to that of a facultative pond, but with an aerobic pond close to the surface and a deeper anaerobic zone. In the present desing only two parallel ponds are considered on the downstream of the anaerobic pond. As stated

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above, the depth of the ponds is taken as 3.5 m inclusive of a 1.0 m zone for sludge in both the ponds.

Table 2. 36 Design calculations of aerated facultative pond after UASBR

DESIGN OF FACULTATIVE WASTE STABILIZATION POND OF 1, 2 AND 5 MLD CAPACITY AFTER UASBR									
S. No.	**Design parameter**	**Design output**							
1	Capacity of the STP	1	MLD		2	MLD		5	MLD
2	Capacity of the STP	1000	m^3/day		2000	m^3/day		5000	m^3/day
3	Capacity of the STP	0.0116	m^3/s		0.0231	m^3/s		0.0579	m^3/s
4	Per capita supply	135	LPCD		135	LPCD		135	LPCD
5	Influent BOD_5 (As per Table 5.4 of CPHEEO Manual - 2013)	27	g /capita per day		27	g /capita per day		27	g /capita per day
6	Influent BOD_5 for Anearobic pond	250	mg/l		250	mg/l		250	mg/l
7	Influent BOD, for Facuiltative pond	87.5	mg/l		87.5	mg/l		87.5	mg/l
8	Effluent BOD_5 (As per Schedule VI of environment (protection) third Amendment Rules, 1993 - for Inland surface water)	30	mg/l		30	mg/l		30	mg/l
9	Average ambient temperature in December	14.5	^oC		14.5	^oC		14.5	^oC
10	Detention time	9	days		9	days		9	days
	Determine effluent soluble BOD_5								
11	BOD removal coefficient K at 20^oC	0.7	day^{-1}		0.7	day^{-1}		0.7	day^{-1}

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12	BOD removal coefficient K at design temperature	0.58	day^{-1}		0.58	day^{-1}		0.58	day^{-1}
13	Soluble BOD_5	14.08	mg/l		14.08	mg/l		14.08	mg/l
	Determine effluent Particulate BOD_5								
14	Suspended Solids in the effluent	50	mg/l		50	mg/l		50	mg/l
15	Particulate BOD_5	15	mg/l		15	mg/l		15	mg/l
	Determine total effluent BOD_5								
16	Total effluent BOD_5	29.08	mg/l		29.08	mg/l		29.08	mg/l
17	Check for the effluent BOD	OK			OK			OK	
18	Efficiency of BOD removal	66.76	%		66.76	%		66.76	%
	Determine dimensions of the aerated facultative pond								
19	Volume required	9000	m^3		18000	m^3		45000	m^3
20	Depth of the pond	3.5	m		3.5	m		3.5	m
21	Pond area required	2571.43	m^2		5142.86	m^2		12857.14	m^2
	Oxygen and Power Requirement								
22	Kg oxygen required to oxidize Kg BOD_5	1.2	KgO_2/ Kg BOD_5		1.2	KgO_2/ Kg BOD_5		1.2	KgO_2/ Kg BOD_5
23	Oxygen required (Standard) per day	69	KgO_2 / day		138	KgO_2 / day		345	KgO_2 / day

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24	Oxygen required (Standard) per hour	2.88	KgO_2 / hour		5.75	KgO_2 / hour		14.37	KgO_2 / hour
25	Oxygenation efficiency in standard conditions (aerator dependent)	1.8	KgO_2 /KWh		1.8	KgO_2 /KWh		1.8	KgO_2 /KWh
26	Field oxygenation efficiency as percentage of standard oxygenration efficiency	60	%		60	%		60	%
27	Oxygenration efficiency in field conditions	1.08	KgO_2 /KWh		1.08	KgO_2 /KWh		1.08	KgO_2 /KWh
28	Tota power required	2.66	KW		5.32	KW		13.31	KW
29	Total power required (say)	3	KW		6	KW		14	KW
30	Power of each aerator	1.5	KW		3	KW		7	KW
31	No. of aerators to be provided	2			2			2	
32	Average power level in the whole pond	0.33	W/m^3		0.33	W/m^3		0.31	W/m^3
33	No. of ponds in parallel	2			2			2	
34	Population equivalent	8818			17637			44092	
35	Area per person (net)	0.29	m^2		0.29	m^2		0.29	m^2
36	Rate of sludge accumulation	0.07	m^3/person /year		0.07	m^3/person /year		0.07	m^3/person/ year
37	Sludge volume in 1 year	617.28	m^3 / year		1234.57	m^3		3086.42	m^3
38	Depth in the cell to accommodate the sludge	1	m		1	m		1	m
39	Volume of sludge in the two cells	2571.42	m^3		5142.86	m^3		12857.14	m^3

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40	Duration of sludge cleaning	4.16	years		4.16	years		4.16	years
41	Area of each pond	1285.71	ha		2571.43	ha		6428.57	ha
42	L/B raio	4	dimensionless		4	dimensionless		4	dimensionless
43	Length L	71.71	m		101.42	m		160.36	m
44	Breadth B	17.93	m		25.35	m		40.09	m
45	Tank L x B X D (Mid-Depth)	72 x 18 x 3.5	m		102 x 25 x 3.5	m		161 x 40 x 3.5	m
46	Length at mid-depth of the tanks	72	m		102	m		161	m
47	Width at mid-depth of the tanks	18	m		25	m		40	m
48	Side slope of pond is 1V :2.0 H	2	m		2	m		2	m
49	Top of embankment above mid depth	2.75	m		2.75	m		2.75	m
50	Tank Bottom below mid-depth	1.75	m		1.75	m		1.75	m
51	Top length	83	m		113	m		172	m
52	Top width	29	m		36	m		51	m
53	Bottom length	65	m		95	m		154	m
54	Bottom width	11	m		18	m		33	m

Green cells: Input variable data to be provided Yellow cells: standard data from CPHEEO manual or other standards Red Cells: Design output

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Table 2. 37 Summary of 1, 2, and 5 MLD Aerated Facultative Ponds after UASBR

Capacity MLD	Component	No. of ponds	Length (m)	Width (B)	Depth (D)
1	Aerated Facultative Pond (Top / Middle / Bottom)	2	83/72/65	29/18/11	3.5
2	Aerated Facultative Pond (Top / Middle / Bottom)	2	113/102/95	36/25/18	3.5
3	Aerated Facultative Pond (Top / Middle / Bottom)	2	172/161/154	51/40/33	3.5

2.13 Design of Secondary Settling Tank

Secondary settling tank is provided in activated sludge process and its variants like SBR, MBBR and extended aeration. The design of 1 MLD, 2 MLD and 5 MLD capacity STPs is presented in this section.

2.13.1 Secondary Settling Tank for ASP and MBBR

The design of secondary settling tank for activated sludge processes and its variates is done by adopting the design parameters given in Table 5.8 of CPHEEO's Manual on Sewerage and Sewage Treatment Systems -2013. The design has been done in a spread sheet for 1 MLD, 2 MLD and 5 MLD capacity STPs. The design outcome is presented in Table 2.38 and the summary is given in Table 2.39.

Table 2. 38 Design calculations for secondary settling tank of Activated sludge Processes

S. No.	Parameter	Reference / Formula	Value	Unit		Value	Unit		Value	Unit
1	Flow Q		1	MLD		2	MLD		5	MLD
2	Flow Q	MLD x 10^6/10^3	1000	m^3/day		2000	m^3/day		5000	m^3/day

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S. No.	Parameter	Reference / Formula	Value	Unit		Value	Unit		Value	Unit
3	Peak factor		2.25			2.25			2.25	
4	Overflow rate for average flow	As per Table 5.8 of CPHEEO manual	15-35	$m^3/m^2/$ day		15-35	$m^3/m^2/$ day		15-35	$m^3/m^2/$ day
5	Overflow rate for peak flow	As per Table 5.8 of CPHEEO manual	40-50	$m^3/m^2/$ day		40-50	$m^3/m^2/$ day		40-50	$m^3/m^2/$ day
6	Solid loading rate for average flow	As per Table 5.8 of CPHEEO manual	70-140	Kg/day $/m^2$		70-140	Kg/day $/m^2$		70-140	Kg/day $/m^2$
7	Solid loading rate for peak flow	As per Table 5.8 of CPHEEO manual	210	Kg/day $/m^3$		210	Kg/day $/m^3$		210	Kg/day $/m^3$
8	Side water depth	As per Table 5.8 of CPHEEO manual	3-3.5	m		3-3.5	m		3-3.5	m
9	Weir loading rate	As per Table 5.8 of CPHEEO manual	185	cum/m /day		185	cum/m /day		185	cum/m/ day
10	MLSS for ASP		3500	mg/l		3500	mg/l		3500	mg/l
11	Adopted surface loading rate	Average flow	20	$m^3/m^2/$ day		20	$m^3/m^2/$ day		20	$m^3/m^2/$ day
12	Surface area required	average Q/surface loading rate	50	m^2		100	m^2		250	m^2
13	Computed surface loading rate	peak factor x average Q / surface area	45	$m^3/m^2/$ day		45	$m^3/m^2/$ day		45	$m^3/m^2/$ day
14	Check for solid loading at average flow	Average Q x MLSS/(srufuce area x 1000)	70	Kg/day $/m^2$		70	Kg/day $/m^2$		70	Kg/day $/m^2$
15	Check for solid loading at peak flow	Peak factor x Average q x MLSS/(Surface area x 1000)	157.5	Kg/day $/m^2$		157.5	Kg/day $/m^2$		157.5	Kg/day $/m^2$
16	Diameter of the tank	SQRT(4 x surface area/π)	7.9808	m		11.286 65	m		17.845 77	m

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S. No.	Parameter	Reference / Formula	Value	Unit		Value	Unit		Value	Unit
17	Adopted diameter of the tank		8	m		11.5	m		18	m
18	Provide detention time of		2	hours		2	hours		2	hours
19	Volume of tank required	Average Q x Detention time /24	83.333	m^3		166.66 67	m^3		416.66 67	m^3
20	Depth required	Volume of tank / surface area	1.6666	m		1.6666 67	m		1.6666 67	m
21	Adopted water depth		2	m		2	m		2	m
22	Provision for sludge		0.3	m		0.3	m		0.3	m
23	Free board		0.4	m		0.4	m		0.4	m
24	Total depth of tank	water depth + sludge depth + free board	2.7	m		2.7	m		2.7	m
25	Check for weir loading rate	average Q / perimeter	39.808 9	m^3/m/d ay		55.386 32	m^3/m/d ay		88.464 26	m^3/m/d ay

Table 2. 39 Summary of secondary settling tank for ASP, SBR and MBBR

S. No.	Capacity	Diameter (m)	Depth (m)
1	1 MLD	8	2.7
2	2 MLD	11.5	2.7
3	5 MLD	18	2.7

2.13.2 Design of Secondary Settling Tank for Extended Aeration Process

The design of secondary settling tank for extended aeration process is done by adopting the design parameters given in Table 5.8 of CPHEEO's Manual on Sewerage and Sewage

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Treatment Systems - 2013. The design has been done in a spread sheet for 1 MLD, 2 MLD and 5 MLD capacity STPs. The design outcome is presented in Table 2.40 and the summary is given in Table 2.41.

Table 2. 40 Design calculations for secondary settling tank of Extended Aeration Process

S. No.	Parameter	Reference / Formula	Value	Unit		Value	Unit		Value	Unit
1	Flow Q		1	MLD		2	MLD		5	MLD
2	Flow Q	MLD x 10^6/10^3	1000	m^3/day		2000	m^3/day		5000	m^3/day
3	Peak factor		2.25			2.25			2.25	
4	Overflow rate for average flow	As per Table 5.8 of CPHEEO manual	8-15	m^3/m^2/ day		8-15	m^3/m^2/ day		8-15	m^3/m^2/ day
5	Overflow rate for peak flow	As per Table 5.8 of CPHEEO manual	25-35	m^3/m^2/ day		25-35	m^3/m^2/ day		25-35	m^3/m^2/ day
6	Solid loading rate for average flow	As per Table 5.8 of CPHEEO manual	25-120	Kg/day /m^2		25-120	Kg/day /m^2		25-120	Kg/day /m^2
7	Solid loading rate for peak flow	As per Table 5.8 of CPHEEO manual	170	Kg/day /m^3		170	Kg/day /m^3		170	Kg/day /m^3
8	Side water depth	As per Table 5.8 of CPHEEO manual	3-4.0	m		3-4.0	m		3-4.0	m
9	Weir loading rate	As per Table 5.8 of CPHEEO manual	185	cum/m /day		185	cum/m /day		185	cum/m/ day
10	MLSS for ASP		3500	mg/l		3500	mg/l		3500	mg/l
11	Adopted surface loading rate	Average flow	13	m^3/m^2/ day		13	m^3/m^2/ day		13	m^3/m^2/ day
12	Surface area required	average Q/surface loading rate	76.923 08	m^2		153.84 62	m^2		384.61 54	m^2
13	Computed surface loading rate	peak factor x average Q / surface area	29.25	m^3/m^2/ day		29.25	m^3/m^2/ day		29.25	m^3/m^2/ day

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S. No.	Parameter	Reference / Formula	Value	Unit		Value	Unit		Value	Unit
14	Check for solid loading at average flow	Average Q x MLSS/(srufface area x 1000)	45.5	Kg/day/m^2		45.5	Kg/day/m^2		45.5	Kg/day/m^2
15	Check for solid loading at peak flow	Peak factor x Average q x MLSS/(Surface area x 1000)	102.375	Kg/day/m^2		102.375	Kg/day/m^2		102.375	Kg/day/m^2
16	Diameter of the tank	SQRT(4 x surface area/π)	9.899049	m		13.99937	m		22.13495	m
17	Adopted diameter of the tank		10	m		14	m		22.5	m
18	Provide detention time of		2	hours		2	hours		2	hours
19	Volume of tank required	Average Q x Detention time /24	83.33333	m^3		166.6667	m^3		416.6667	m^3
20	Depth required	Volume of tank / surface area	1.083333	m		1.083333	m		1.083333	m
21	Adopted water depth		2	m		2	m		2	m
22	Provision for sludge		0.3	m		0.3	m		0.3	m
23	Free board		0.4	m		0.4	m		0.4	m
24	Total depth of tank	water depth + sludge depth + free board	2.7	m		2.7	m		2.7	m
25	Check for weir loading rate	average Q / perimeter	31.84713	m^3/m/day		45.49591	m^3/m/day		70.77141	m^3/m/day

Table 2. 41 Summary of secondary settling tank for Extended Aeration Process

S. No.	Capacity	Diameter (m)	Depth (m)
1	1 MLD	10	2.7
2	2 MLD	14	2.7

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3	5 MLD	22.5	2.7

2.14 Design of Chlorination Tank

Disinfection of the treated Used Water is done after the secondary settling tank. As per Section 5.9.2 of CPHEEO's manual on Sewerage and Sewage Treatment Systems - 2013, 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. 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.

2.14.1. Design of Chlorination tank for 2 MLD

The design process of chlorination tank is very simple. The size of the tank is obtained by considering the detention period as 30 minutes.

Flow = 2 MLD

= 2000 m^3/day

Detention period = 30 minutes

Volume of tank = $2 \times 10^6 \times 0.5 / (10^3 \times 24)$

= 41.67 m^3

Let depth be = 2.7 m

Free board = 0.3 m

Total depth of tank = 3.0 m

Area of tank = Volume / depth

= 41.67 / 3.0

= 13.89 m^2

Required side of the tank = $\sqrt{13.89}$

= 3.73 m

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Hence provide a tank of size 4.0 x 4.0 x 3.0 m

The summary of design of chlorination tank for 1 MLD, 2 MLD and 3 MLD capacity STPs is presented in Table 2.42.

Table 2. 42 Summary of chlorination tank

S. No.	Capacity	Length (m)	Breadth (m)	Depth (m)
1	1 MLD	3.3	3.3	2.3
2	2 MLD	4.0	4.0	3.0
3	5 MLD	6.0	6.0	3.3

2.15 Summary of Tank Sizes

The design elements consist of wet well, coarse and medium screens, grit chamber, reactor, sedimentation tank and chlorination tank. The summary of the study of eight technologies / combination of technologies in terms of the tank sizes is summarized in Table 2.43.

Table 2. 43 Consolidated statement of tank sizes

Technology	Component	Capacity	No.	Dia (m)	Length (m)	Width (m)	Depth (m)	Other
All	Wet Well	1 MLD		2.5			6.5	2 W + 1 SB 1 KW pump
		2 MLD		3.5			6.5	2 W + 1 SB 2 KW pump
		5 MLD		4.5			6.5	2 W + 1 SB 5 KW pump
All	Coarse Screen	1 MLD	2		2.0	0.30	0.6	
		2 MLD	2		3.0	0.45	0.6	
		5 MLD	2		6.5	1.00	0.6	

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Technology	Component	Capacity	No.	Dia (m)	Length (m)	Width (m)	Depth (m)	Other
All	Medium Screen	1 MLD	2		3.5	0.4	0.6	
		2 MLD	2		4.5	0.5	0.7	
		5 MLD	2		8.0	0.9	1.0	
All	Grit Chamber	1 MLD	1		12	0.3	0.9	
		2 MLD	1		12	0.6	0.9	
		5 MLD	1		12	0.8	1.20	
Waste Stabilization Pond	Anaerobic Pond	1 MLD	1		46/32/22	30/16/6	6	Values given under L and B of a pond refer to Top / Mid-depth / Bottom dimensions of the tank. Side slope if 1 V : 2 H
		2 MLD	1		59/45/35	36.5/22.5 / 12.5	6	
		5 MLD	2		60/50/40	39/25/15	6	
	Facultative pond Primary	1 MLD	2		116/108/ 104	35/27/23	2	
		2 MLD	2		160/152/ 148	46/38/34	2	
		5 MLD	2		248/240/ 236	68/60/56	2	
	Facultative pond Secondary	1 MLD	1		115/108/ 105	34/27/24	1.5	
		2 MLD	1		159/152/ 149	45/38/35	1.5	
		5 MLD	1		247/240/ 237	67/60/57	1.5	
Anaerobic Pond + Aerated Facultative Ponds	Aerated Facultative Ponds	1 MLD	2		97/86/79	33/22/15	3.5	
		2 MLD	2		133/122/ 115	41.5/30.5 /23.5	3.5	
		5 MLD	2		204/193/ 186	59/48/41	3.5	

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Technology	Component	Capacity	No.	Dia (m)	Length (m)	Width (m)	Depth (m)	Other
Anaerobic Baffled Reactor + Constructed Wetlands	Anaerobic Settler	1 MLD	4		13.5	5.0	1.5	Transformer + Miscellaneous
		2 MLD	8		13.5	5.0	1.5	
		5 MLD	20		13.5	5.0	1.5	
	ABR	1 MLD	4		6	23.5	1.5	
		2 MLD	8		6	23.5	1.5	
		5 MLD	20		6	23.5	1.5	
	Constructed wetlands	1 MLD	4		87	15.0	0.6	
		2 MLD	8		87	15.0	0.6	
		5 MLD	20		87	15.0	0.6	
Upflow Anaerobic Sludge Blanket + Waste Stabilization Ponds	UASBR	1 MLD	1		11.5	5.6	7.0	Gas Balloon, sludge tank, centrifuge, filter press, transformer + Miscellaneous
		2 MLD	1		15.9	8.2	7.0	
		5 MLD	1		25.0	12.5	7.0	
	Facultative Pond Primary	1 MLD	2		97/89//85	30/22/18	2	
		2 MLD	2		134/126/ 122	40/32/28	2	
		5 MLD	2		208/200/ 196	58/50/46	2	
	Facultative Pond Secondary	1 MLD	1		96/89/86	29/22/19	1.5	
		2 MLD	1		133/126/ 123	39/32/29	1.5	
		5 MLD	1		207/200/ 197	57/50/47	1.5	
UASBR + Aerated Fcultative Ponds	Aerated Facultative Ponds	1 MLD	2		83/72/65	29/18/11	3.5	
		2 MLD	2		113/102/ 95	36/25/18	3.5	
		5 MLD	2		172/161/ 154	51/40/33	3.5	

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Technology	Component	Capacity	No.	Dia (m)	Length (m)	Width (m)	Depth (m)	Other
Upflow Anaerobic Sludge Blanket + Activated Sludge Process	UASBR	1 MLD	1		11.5	5.6	7.0	Gas Balloon, Blower room, sludge tank, centrifuge, filter press, transformer, generator + Miscellaneous
		2 MLD	1		15.9	8.2	7.0	
		5 MLD	1		25.0	12.5	7.0	
	ASP	1 MLD	1		6.25	6.25	6.0	
		2 MLD	1		8.75	8.75	6.0	
		5 MLD	1		14.0	14.0	6.0	
	Secondary Settling Tank	1 MLD	1	8			2.7	
		2 MLD	1	11.5			2.7	
		5 MLD	1	18			2.7	
Sequencing Batch Reactor	Reactor	1 MLD	2		14.5	14.5	6.3	Blower Room, Sludge Tank, Centrifuge, Filter Press, Transformer, Generator + Miscellaneous
		2 MLD	2		20.5	20.5	6.3	
		5 MLD	2		32.5	32.5	6.3	
Extended Aeration Process	Reactor	1 MLD	1		13.5	13.5	6.00	Blower Room, Sludge Tank, Centrifuge, Filter Press, Transformer, Generator + Miscellaneous
		2 MLD	1		19.25	19.25	6.00	
		5 MLD	1		30.25	30.25	6.00	
	Secondary Settling Tank	1 MLD	1	10			2.7	
		2 MLD	1	14			2.7	
		5 MLD	1	22.5			2.7	
Moving Bed Biofilm Reactor	Reactor	1 MLD	1	8.25			4.5	Blower Room, Sludge Tank, Centrifuge, Filter Press,
		2 MLD	1	11.75			4.5	

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Technology	Component	Capacity	No.	Dia (m)	Length (m)	Width (m)	Depth (m)	Other
		5 MLD	1	18.5	``		4.5	Transformer, Generator + Miscellaneous
	Secondary Settling Tank	1 MLD	1	8			2.7	
		2 MLD	1	11.5			2.7	
		5 MLD	1	18			2.7	
Trickling Filter	Primary Settling Tank	1 MLD	1	8.75			3.25	Recirculation = 0.2 l/m^2.s Flushing dose rate 300 mm/pass Operation dose 46 mm/pass Pump for 650 l/s with head of 8.5 m
		2 MLD	1	12.5			3.25	
		5 MLD	1	19.5			3.25	
	Bio-Tower	1 MLD	1	7.0			6.0	
		2 MLD	1	10.0			6.0	
		5 MLD	1	16.0			6.0	
	Secondary Settling Tank	1 MLD	1	8.0			2.70	
		2 MLD	1	11.5			2.70	
		5 MLD	1	18.0			2.70	
All	Chlorine contact tank	1 MLD	1		3.3	3.3	2.3	Chlorine pump 1 W + 1 SB
		2 MLD	1		4	4	3	
		5 MLD	1		6	6	3.3	

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CHAPTER – 3

COST ESTIMATES & LIFE CYCLE COST ANALYSIS

3.1 Project Costing

The cost estimation for a sewage treatment plant (STP) for municipal sewage can vary significantly based on various factors such as; capacity of the STP, technology selected, regulatory requirements on effluent, level of automation, O & M cost etc.

The project cost can be divided into two heads, CapEx ad OpEx. Understanding the differentiation between CapEx and OpEx is crucial for financial planning and management. For sewage treatment plants, the CapEx constitutes the initial investment needed to build the infrastructure, while the OpEx represents the ongoing operational costs necessary to keep the plant functioning efficiently.

It's important for organizations to evaluate both CAPEX and OPEX comprehensively when planning and budgeting for a sewage treatment plant, as they have different implications on financial management, project feasibility, and long-term sustainability.

3.1.1 Capital Expenditure (CapEx)

CapEx refers to the initial investment or expenses incurred to acquire, upgrade, or improve fixed assets, such as buildings, equipment, land, and infrastructure required for the sewage treatment plant's construction.

In the context of a sewage treatment plant, CapEx includes costs like land acquisition, construction of treatment facilities, purchasing equipment (pumps, tanks, filters, etc.), installation, engineering and design fees, permits, and other one-time expenses. CapEx is classified as a non-recurring expenditure since it is a onetime investment.

CAPEX is typically a significant upfront cost that is invested at the beginning of the project or during its expansion or enhancement phases.

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3.1.2. Operating Expenditure (OpEx)

OpEx represents the day-to-day operational costs incurred in running and maintaining the sewage treatment plant after it's constructed. OpEx includes regular expenses such as labour wages, utility bills (electricity, water), chemical and material costs for treatment processes, routine maintenance, repair and replacement of equipment, administrative costs, and compliance-related expenses. Unlike CapEx, OpEx is recurring and ongoing throughout the life cycle of the sewage treatment plant.

3.1.3. Basis of Costing

The following inputs are required for detailed cost estimation of the STP project:

i. Layout of the plant
ii. Detailed design of all STP components as well as utilities like office room, laboratory, wash room etc.
iii. Specifications of the components
iv. Latest SSR of Government of Telangana. For items which are not included in the SSR, such as electro mechanical equipment, the costing is done based on market rates obtained by calling quotations from suppliers.
v. For estimate of O & M expenditure, including ward and watch, the CPHEEO manual is relied upon.

3.2. CapEx and OpEx of the Technologies in the Present Study

The CapEx is obtained by including the cost of civil structures including administrative building, laboratory with equipment as specified in CPHEEO manual, panel room, wash room, watchman's room, electromechanical equipment like pumps, motors, blowers, transformer etc. and finally the land cost.

The OpEx is obtained by summing up the recurring expenditure towards establishment, energy consumption, consumables and fuel, laboratory chemicals, repairs, renewals and minor replacements.

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3.3 Life Cycle Cost Analysis (LCCA)

Life Cycle Cost Analysis (LCCA) is a methodology used to assess the total costs associated with the project over its entire life span. In the context of a sewage treatment plant, conducting an LCCA involves evaluating all costs associated with the plant from its inception, through construction, operation, maintenance, and eventual decomMissioning.

The identified costs to be considered in the LCCA are as follows:

i. CapEx including land cost
ii. OpEx including salaries
iii. Disposal or decomMissioning cost at the end of the life cycle.
iv. Time frame and discount rate. The time frame is related to the expected useful life of the project, which is 15 years in the present study. The discount rate is applied to account for the time value of money, as future costs are typically less valuable than the present costs.

The analysis consist of calculating the present value of all costs over the plant's life cycle, including CapEx and OpEx. This involves discounting future costs to their present value to facilitate meaningful comparisons.

1. The life cycle cost is calculated for 15 years period.
2. The CapEx and OpEx are obtained from Table 4.5 for 2038 design as per CPHEEO norms and Table 4.6 as per SBM – U 2.0 norms.
3. The area required for the STP is obtained from the layout plan.
4. Zero land cost implies availability of government land for construction of the STP.
5. Manpower cost is obtained by assuming minimum staff as per the guidelines given in Table 5.3 of Part C of the CPHEEO manual.
6. The power cost is taken as RS. 8.00 per KWh.
7. The interest rate is taken as 10% per anum.
8. The annual escalation rate for O & M expenses is taken as 10%.

The outcome of the LCC analysis is presented in Tables 3.1 to b 3.17 and figure 3.1 to 3.4.

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Table 3. 1 Per MLD CaPex of the ten technologies chosen for evaluation

STP Cost Per MLD in Cores of Rs.											
S.No.	STP Capacity	TECHNOLOGY									
		Waste Stabilization Ponds(WSP)	Anarobic Baffled Reactor + Consturcted Wet Land (ABR + CWL)	Upflow Anaerobic Sludge Blanket + Waste Stabilization Ponds (UASBR + WSP)	Upflow Anaerobic Sludge Blanket + Activated Sludge Process (UASBR + ASP)	Sequencing Batch Reactor (SBR)	Extended Aeration Process (EAP)	Moving Bed Biofilm Reactor (MBBR)	Anaerobic Pond + Aerated Facultative Pond (ANP+AFP)	Up Flow - Anaerobic Sludge Blanket Reactor+ Aerated Faculative Pond (UASBR + AFP)	Bio Tower (Tricking Filter)
		(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1	1 MLD	1.36	2.48	1.66	3.33	5.09	4.40	2.76	1.51	1.75	1.60
2	2 MLD	1.02	2.28	1.17	2.16	3.48	2.64	1.67	1.21	1.20	1.43
3	5 MLD	0.77	2.22	0.78	1.32	2.33	1.72	1.10	1.01	0.89	0.98

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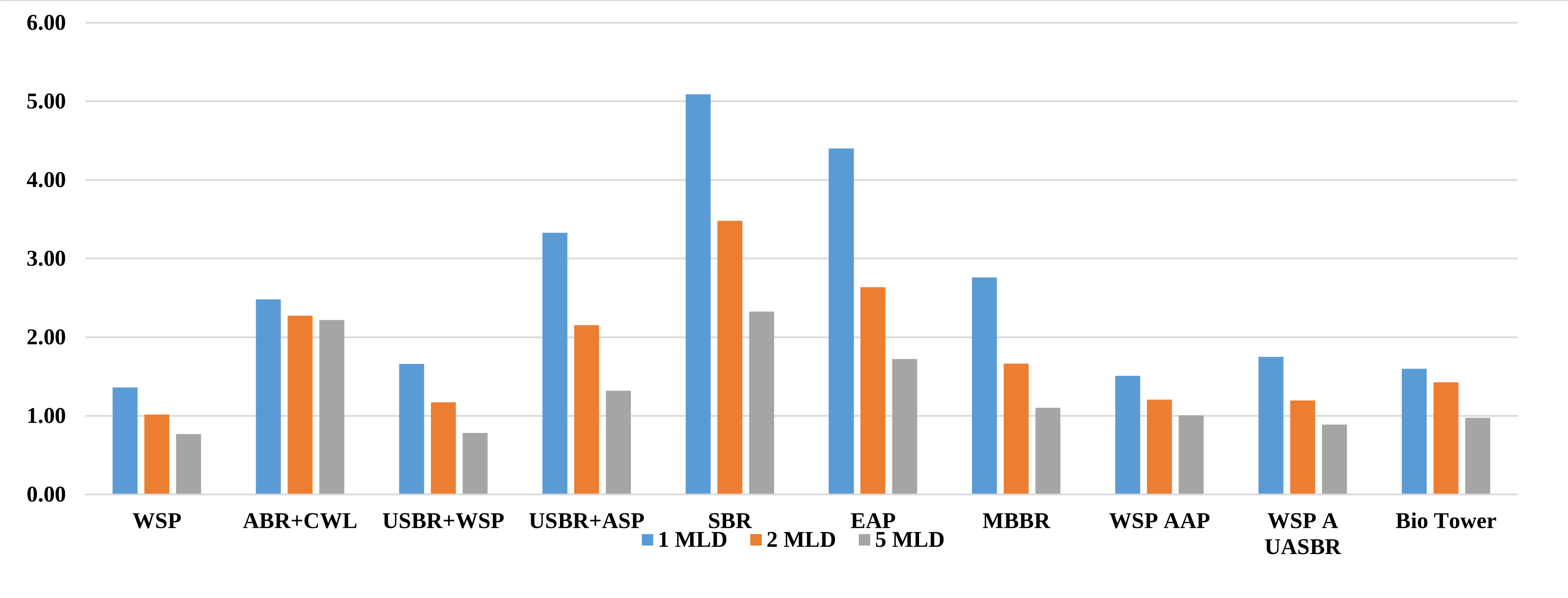

WSP : Waste Stabilization Pond, **ABR :** Anaerobic Baffled Reactor, **CWL :** Constructed Wet Land, **UASBR :** Upflow Anaerobic Sludge Blanket Reactor,

Figure 3. 1 Per MLD Capital Cost of the ten technologies chosen for evaluation

The capital cost is steadily decreasing across all technologies with increase of capacity from 1 MLD to 5 MLD
The percentage reduction from 1 MLD to 5 MLD is as follows

Technology	WSP	ABR + CWL	UASBR + WSP	UASBR + ASP	SBR	EAP	MBBR	ANP + AFP	UASBR + AFP	Bio-Tower
Percentage reduction in capital cost from 1 to 5 MLD	44	10	53	60	54	61	60	33	49	39

Table 3. 2 LCCA for 1 MLD plants with 0 land cost

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Life Cycle Cost (LCC) - Parameters - 1.0 MLD STP Capacity

S. No.	Technologies	Capital Cost (Rs. In Lakhs)	Cost (Rs. In Lakhs)							
			Land		O&M Cost					LCC at the end of 15 Years
			Required (In Acres)	Cost @ 0 Lakhs per Acre	Manpower	Power	Others	Total		
								Per Annum	15 Years @10% escalation per annum	
1	Waste Stabilization Ponds (WSP)	136.00	5.04	0.00	5.88	0.17	5.00	11.05	281.78	849.88
2	Anaerobic Bafled Reactor (ABR) + Constructed Wet Lands (CWL)	248.00	3.7	0.00	5.88	0.17	5.00	11.05	281.78	1317.73
3	Up Flow - Anaerobic Sludge Blanket Reactor+ WSP (UASBR + WSP)	166.00	4.10	0.00	8.47	0.23	6.10	14.80	377.40	1070.82
4	Up Flow - Anaerobic Sludge Blanket Reactor+ ASP (UASBR + ASP)	333.00	0.51	0.00	18.41	3.67	6.10	28.18	718.59	2109.61
5	Sequence Batch Reactor (SBR)	509.00	0.50	0.00	18.41	4.49	13.12	36.02	918.51	3044.73
6	Extended Aeration Process (EAP)	440.00	0.52	0.00	18.41	5.42	13.12	36.95	942.23	2780.21
7	Moving Bed Bio Reactor (MBBR)	276.00	0.43	0.00	18.41	6.53	13.12	38.06	970.53	2123.45
8	Anaerobic Pond + Aerated Facultative Pond (AP+AFP)	151.00	4.04	0.00	5.88	0.17	5.00	11.05	281.78	912.54
9	Up Flow - Anaerobic Sludge Blanket Reactor+ Aerated Faculative Pond (UASBR + AFP)	175.00	3.38	0.00	8.47	0.23	6.10	14.80	377.40	1108.42
10	Bio Tower (Tricking Filter)	160.00	0.43	0.00	18.41	7.84	13.12	39.37	1003.83	1672.19

Note: Manpower considered for WSP, ABR+CW = 1 ITI Plumber & Watchman / unskilled labour, UASB+WSP= 1 ITI Plumber, Watchman / Unskilled labour + Skilled labour (3 No.), UASB+ASP= 0.5 (Junior Engineer), 1 ITI Plumber + 1 semi skilled + 1 unskilled + 1 Watchman + 1 Chemist, *Hypochlorite= 5 mg/L =5 Kg/Million Litre*

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Table 3. 3 LCCA for 2 MLD plants with 0 land cost

Life Cycle Cost (LCC) - Parameters - 2.0 MLD STP Capacity

S. No.	Technologies	Capital Cost (Rs. In Lakhs)	Cost (Rs. In Lakhs)								
			Land		O&M Cost					LCC at the end of 15 Years
			Required (In Acres)	Cost@0Lakhs per Acre	Manpower	Power	Others	Total		
								Per Annum	15 Years (@10% escalation per annum)	
1	Waste Stabilization Ponds (WSP)	203.00	8.78	0.00	5.88	0.33	5.28	11.49	293.00	1140.98
2	Anaerobic Bafled Reactor (ABR) + Constructed Wet Lands (CWL)	455.00	7	0.00	5.88	0.33	5.28	11.49	293.00	2193.64
3	Up Flow - Anaerobic Sludge Blanket Reactor+ WSP (UASB + WSP)	234.00	7.90	0.00	8.47	7.34	10.18	25.99	662.75	1640.22
4	Up Flow - Anaerobic Sludge Blanket Reactor+ ASP (UASB + ASP)	431.00	0.64	0.00	18.41	7.34	10.18	35.93	916.22	2716.61
5	Sequence Batch Reactor (SBR)	696.00	0.55	0.00	18.41	8.98	24.20	51.59	1315.55	4222.91
6	Extended Aeration Process (EAP)	527.00	0.69	0.00	18.41	10.84	24.20	53.45	1362.98	3564.38
7	Moving Bed Bio Reactor (MBBR)	333.00	0.56	0.00	18.41	13.06	24.20	55.67	1419.59	2810.61
8	Anaerobic Pond + Aerated Facultative Pond (AP+AFP)	241.00	6.96	0.00	5.88	0.33	5.28	11.49	293.00	1299.71
9	Up Flow - Anaerobic Sludge Blanket Reactor+ Aerated Faculative Pond (UASBR + AFP)	239.00	6.58	0.00	8.47	7.34	10.18	25.99	662.75	1661.11
10	Bio Tower (Tricking Filter)	285.00	0.5	0.00	18.41	13.06	24.20	55.67	1419.59	2610.10

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Table 3. 4 LCCA for 5 MLD plants with 0 land cost

Life Cycle Cost (LCC) - Parameters - 5.0 MLD STP Capacity

S. No.	Technologies	Capital Cost (Rs. In Lakhs)	Cost (Rs. In Lakhs)							
			Land		O&M Cost					LCC at the end of 15 Years
			Required (In Acres)	Cost@0Lakhs per Acre	Manpower	Power	Others	Total		
								Per Annum	15 Years (@10% escalation per annum)	
1	Waste Stabilization Ponds (WSP)	384.00	20.25	0.00	5.88	0.83	6.10	12.81	326.66	1930.72
2	Anaerobic Bafled Reactor (ABR) + Constructed Wet Lands (CWL)	1110.00	16.96	0.00	5.88	0.83	6.10	12.81	326.66	4963.40
3	Up Flow - Anaerobic Sludge Blanket Reactor+ WSP (UASB + WSP)	391.00	18.20	0.00	8.47	18.35	22.38	49.20	1254.60	2887.90
4	Up Flow - Anaerobic Sludge Blanket Reactor+ ASP (UASB + ASP)	659.00	0.83	0.00	18.41	18.35	22.38	59.14	1508.07	4260.88
5	Sequence Batch Reactor (SBR)	1163.00	0.86	0.00	18.41	22.44	57.43	98.28	2506.14	7364.28
6	Extended Aeration Process (EAP)	861.00	1.13	0.00	18.41	27.11	57.43	102.95	2625.23	6221.84
7	Moving Bed Bio Reactor (MBBR)	552.00	0.8	0.00	18.41	32.66	57.43	108.50	2766.75	5072.59
8	Anaerobic Pond + Aerated Facultative Pond (AP+AFP)	503.00	16.09	0.00	5.88	0.83	6.10	12.81	326.66	2427.81
9	Up Flow - Anaerobic Sludge Blanket Reactor+ Aerated Faculative Pond (UASBR + AFP)	444.00	15.30	0.00	8.47	18.35	22.38	49.20	1254.60	3109.30
10	Bio Tower (Tricking Filter)	488.00	0.76	0.00	18.41	32.66	57.43	108.50	2766.75	4805.25

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Table 3. 5 LCCA for 1 MLD plants with Rs. 10 lakhs per acre land cost

Life Cycle Cost (LCC) - Parameters - 1.0 MLD STP Capacity

S. No.	Technologies	Capital Cost (Rs. In Lakhs)	Cost (Rs. In Lakhs)							
			Land		O&M Cost					LCC at the end of 15 Years
			Required (In Acres)	Cost@10Lakhs per Acre	Manpower	Power	Others	Total		
								Per Annum	15 Years (@10% escalation per annum)	
1	Waste Stabilization Ponds (WSP)	136.00	5.04	50.40	5.88	0.17	5.00	11.05	281.78	1060.41
2	Anaerobic Bafled Reactor (ABR) + Constructed Wet Lands (CWL)	248.00	3.7	37.00	5.88	0.17	5.00	11.05	281.78	1472.29
3	Up Flow - Anaerobic Sludge Blanket Reactor+ WSP (UASB + WSP)	166.00	4.10	41.00	8.47	3.67	6.10	18.24	465.12	1329.81
4	Up Flow - Anaerobic Sludge Blanket Reactor+ ASP (UASB + ASP)	333.00	0.51	5.10	18.41	3.67	6.10	28.18	718.59	2130.92
5	Sequence Batch Reactor (SBR)	509.00	0.50	5.00	18.41	4.49	13.12	36.02	918.51	3065.62
6	Extended Aeration Process (EAP)	440.00	0.52	5.20	18.41	5.42	13.12	36.95	942.23	2801.94
7	Moving Bed Bio Reactor (MBBR)	276.00	0.43	4.30	18.41	6.53	13.12	38.06	970.53	2141.41
8	Anaerobic Pond + Aerated Facultative Pond (AP+AFP)	151.00	4.04	40.40	5.88	0.17	5.00	11.05	281.78	1081.30
9	Up Flow - Anaerobic Sludge Blanket Reactor+ Aerated Faculative Pond (UASBR + AFP)	175.00	3.38	33.80	8.47	3.67	6.10	18.24	465.12	1337.33
10	Bio Tower (Tricking Filter)	160.00	0.43	4.30	18.41	7.84	13.12	39.37	1003.83	1690.15

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Table 3. 6 LCCA for 2 MLD plants with Rs. 10 lakhs per acre land cost

Life Cycle Cost (LCC) - Parameters - 2.0 MLD STP Capacity

S. No	Technologies	Capital Cost (Rs. In Lakhs)	Cost (Rs. In Lakhs)							
			Land		O&M Cost					LCC at the end of 15 Years
			Required (In Acres)	Cost@10Lakhs per Acre	Manpower	Power	Others	Total		
								Per Annum	15 Years (@10% escalation per annum)	
1	Waste Stabilization Ponds (WSP)	203.00	8.78	87.80	5.88	0.33	5.28	11.49	293.00	1507.74
2	Anaerobic Bafled Reactor (ABR) + Constructed Wet Lands (CWL)	455.00	7	70.00	5.88	0.33	5.28	11.49	293.00	2486.05
3	Up Flow - Anaerobic Sludge Blanket Reactor+ WSP (UASB + WSP)	234.00	7.90	79.00	8.47	7.34	10.18	25.99	662.75	1970.22
4	Up Flow - Anaerobic Sludge Blanket Reactor+ ASP (UASB + ASP)	431.00	0.64	6.40	18.41	7.34	10.18	35.93	916.22	2743.34
5	Sequence Batch Reactor (SBR)	696.00	0.55	5.50	18.41	8.98	24.20	51.59	1315.55	4245.88
6	Extended Aeration Process (EAP)	527.00	0.69	6.90	18.41	10.84	24.20	53.45	1362.98	3593.21
7	Moving Bed Bio Reactor (MBBR)	333.00	0.56	5.60	18.41	13.06	24.20	55.67	1419.59	2834.00
8	Anaerobic Pond + Aerated Facultative Pond (AP+AFP)	241.00	6.96	69.60	5.88	0.33	5.28	11.49	293.00	1590.45
9	Up Flow - Anaerobic Sludge Blanket Reactor+ Aerated Faculative Pond (UASBR + AFP)	239.00	6.58	65.80	8.47	7.34	10.18	25.99	662.75	1935.97
10	Bio Tower (Tricking Filter)	285.00	0.5	5.00	18.41	13.06	24.20	55.67	1419.59	2630.99

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Table 3. 7 LCCA for 5 MLD plants with Rs. 10 lakhs per acre land cost

Life Cycle Cost (LCC) - Parameters - 5.0 MLD STP Capacity

S. No	Technologies	Capital Cost (Rs. In Lakhs)	Cost (Rs. In Lakhs)							
			Land		O&M Cost					LCC at the end of 15 Years
			Required (In Acres)	Cost@10Lakhs per Acre	Manpower	Power	Others	Total		
								Per Annum	15 Years (@10% escalation per annum)	
1	Waste Stabilization Ponds (WSP)	384.00	20.25	202.50	5.88	0.83	6.10	12.81	326.66	2776.61
2	Anaerobic Bafled Reactor (ABR) + Constructed Wet Lands (CWL)	1110.00	16.96	169.60	5.88	0.83	6.10	12.81	326.66	5671.86
3	Up Flow - Anaerobic Sludge Blanket Reactor+ WSP (UASB + WSP)	391.00	18.20	182.00	8.47	18.35	22.38	49.20	1254.60	3648.16
4	Up Flow - Anaerobic Sludge Blanket Reactor+ ASP (UASB + ASP)	659.00	0.83	8.30	18.41	18.35	22.38	59.14	1508.07	4295.55
5	Sequence Batch Reactor (SBR)	1163.00	0.86	8.60	18.41	22.44	57.43	98.28	2506.14	7400.20
6	Extended Aeration Process (EAP)	861.00	1.13	11.30	18.41	27.11	57.43	102.95	2625.23	6269.04
7	Moving Bed Bio Reactor (MBBR)	552.00	0.8	8.00	18.41	32.66	57.43	108.50	2766.75	5106.01
8	Anaerobic Pond + Aerated Facultative Pond (AP+AFP)	503.00	16.09	160.90	5.88	0.83	6.10	12.81	326.66	3099.93
9	Up Flow - Anaerobic Sludge Blanket Reactor+ Aerated Faculative Pond (UASBR + AFP)	444.00	15.30	153.00	8.47	18.35	22.38	49.20	1254.60	3748.42
10	Bio Tower (Tricking Filter)	488.00	0.76	7.60	18.41	32.66	57.43	108.50	2766.75	4836.99

Table 3. 8 LCCA for 1 MLD plants with Rs. 25 lakhs per acre land cost

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Life Cycle Cost (LCC) - Parameters - 1.0 MLD STP Capacity

S. No	Technologies	Capital Cost (Rs. In Lakhs)	Cost (Rs. In Lakhs)							
			Land		O&M Cost					LCC at the end of 15 Years
			Required (In Acres)	Cost@25Lakhs per Acre	Manpower	Power	Others	Total		
								Per Annum	15 Years (@10% escalation per annum)	
1	Waste Stabilization Ponds (WSP)	136.00	5.04	126.00	5.88	0.17	5.00	11.05	281.78	1376.21
2	Anaerobic Bafled Reactor (ABR) + Constructed Wet Lands (CWL)	248.00	3.7	92.50	5.88	0.17	5.00	11.05	281.78	1704.13
3	Up Flow - Anaerobic Sludge Blanket Reactor+ WSP (UASB + WSP)	166.00	4.10	102.50	8.47	0.23	6.10	14.80	377.40	1498.99
4	Up Flow - Anaerobic Sludge Blanket Reactor+ ASP (UASB + ASP)	333.00	0.51	12.75	18.41	3.67	6.10	28.18	718.59	2162.87
5	Sequence Batch Reactor (SBR)	509.00	0.50	12.50	18.41	4.49	13.12	36.02	918.51	3096.94
6	Extended Aeration Process (EAP)	440.00	0.52	13.00	18.41	5.42	13.12	36.95	942.23	2834.52
7	Moving Bed Bio Reactor (MBBR)	276.00	0.43	10.75	18.41	6.53	13.12	38.06	970.53	2168.36
8	Anaerobic Pond + Aerated Facultative Pond (AP+AFP)	151.00	4.04	101.00	5.88	0.00	5.00	10.88	277.44	1330.11
9	Up Flow - Anaerobic Sludge Blanket Reactor+ Aerated Faculative Pond (UASBR + AFP)	175.00	3.38	84.50	8.47	0.23	6.10	14.80	377.40	1461.40
10	Bio Tower (Tricking Filter)	160.00	0.43	10.75	18.41	7.84	13.12	39.37	1003.83	1717.10

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Table 3. 9 LCCA for 2 MLD plants with Rs. 25 lakhs per acre land cost

Life Cycle Cost (LCC) - Parameters - 2.0 MLD STP Capacity

S. No	Technologies	Capital Cost (Rs. In Lakhs)	Cost (Rs. In Lakhs)							
			Land		O&M Cost					LCC at the end of 15 Years
								Total		
			Required (In Acres)	Cost@25Lakhs per Acre	Manpower	Power	Others	Per Annum	15 Years (@10% escalation per annum)	
1	Waste Stabilization Ponds (WSP)	203.00	8.78	219.50	5.88	0.33	5.28	11.49	293.00	2057.88
2	Anaerobic Bafled Reactor (ABR) + Constructed Wet Lands (CWL)	455.00	7	175.00	5.88	0.33	5.28	11.49	293.00	2924.66
3	Up Flow - Anaerobic Sludge Blanket Reactor+ WSP (UASB + WSP)	234.00	7.90	197.50	8.47	7.34	10.18	25.99	662.75	2465.23
4	Up Flow - Anaerobic Sludge Blanket Reactor+ ASP (UASB + ASP)	431.00	0.64	16.00	18.41	7.34	10.18	35.93	916.22	2783.44
5	Sequence Batch Reactor (SBR)	696.00	0.55	13.75	18.41	8.98	24.20	51.59	1315.55	4280.35
6	Extended Aeration Process (EAP)	527.00	0.69	17.25	18.41	10.84	24.20	53.45	1362.98	3636.44
7	Moving Bed Bio Reactor (MBBR)	333.00	0.56	14.00	18.41	13.06	24.20	55.67	1419.59	2869.09
8	Anaerobic Pond + Aerated Facultative Pond (AP+AFP)	241.00	6.96	174.00	5.88	0.00	5.28	11.16	284.58	2018.14
9	Up Flow - Anaerobic Sludge Blanket Reactor+ Aerated Faculative Pond (UASBR + AFP)	239.00	6.58	164.50	8.47	7.34	10.18	25.99	662.75	2348.26
10	Bio Tower (Tricking Filter)	285.00	0.5	12.50	18.41	13.06	24.20	55.67	1419.59	2662.32

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Table 3. 10 LCCA for 5 MLD plants with Rs. 25 lakhs per acre land cost

Life Cycle Cost (LCC) - Parameters - 5.0 MLD STP Capacity

S. No.	Technologies	Capital Cost (Rs. In Lakhs)	Cost (Rs. In Lakhs)							
			Land		O&M Cost					LCC at the end of 15 Years
			Required (In Acres)	Cost@25Lakhs per Acre	Manpower	Power	Others	Total		
								Per Annum	15 Years (@10% escalation per annum)	
1	Waste Stabilization Ponds (WSP)	384.00	20.25	506.25	5.88	0.83	6.10	12.81	326.66	4045.45
2	Anaerobic Bafled Reactor (ABR) + Constructed Wet Lands (CWL)	1110.00	16.96	424.00	5.88	0.83	6.10	12.81	326.66	6734.55
3	Up Flow - Anaerobic Sludge Blanket Reactor+ WSP (UASB + WSP)	391.00	18.20	455.00	8.47	18.35	22.38	49.20	1254.60	4788.55
4	Up Flow - Anaerobic Sludge Blanket Reactor+ ASP (UASB + ASP)	659.00	0.83	20.75	18.41	18.35	22.38	59.14	1508.07	4347.55
5	Sequence Batch Reactor (SBR)	1163.00	0.86	21.50	18.41	22.44	57.43	98.28	2506.14	7454.09
6	Extended Aeration Process (EAP)	861.00	1.13	28.25	18.41	27.11	57.43	102.95	2625.23	6339.84
7	Moving Bed Bio Reactor (MBBR)	552.00	0.8	20.00	18.41	32.66	57.43	108.50	2766.75	5156.14
8	Anaerobic Pond + Aerated Facultative Pond (AP+AFP)	503.00	16.09	402.25	5.88	0.00	6.10	11.98	305.49	4086.94
9	Up Flow - Anaerobic Sludge Blanket Reactor+ Aerated Faculative Pond (UASBR + AFP)	444.00	15.30	382.50	8.47	18.35	22.38	49.20	1254.60	4707.10
10	Biotower	488.00	0.76	19.00	18.41	32.66	57.43	108.50	2766.75	4884.61

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Table 3. 11 LCCA for 1 MLD plants with Rs. 50 lakhs per acre land cost

Life Cycle Cost (LCC) - Parameters - 1.0 MLD STP Capacity

S. No	Technologies	Capital Cost (Rs. In Lakhs)	Cost (Rs. In Lakhs)							
			Land		O&M Cost					LCC at the end of 15 Years
								Total		
			Required (In Acres)	Cost@50Lakhs per Acre	Manpower	Power	Others	Per Annum	15 Years (@10% escalation per annum)	
1	Waste Stabilization Ponds (WSP)	136.00	5.04	252.00	5.88	0.17	5.00	11.05	281.78	1902.55
2	Anaerobic Bafled Reactor (ABR) + Constructed Wet Lands (CWL)	248.00	3.7	185.00	5.88	0.17	5.00	11.05	281.78	2090.52
3	Up Flow - Anaerobic Sludge Blanket Reactor+ WSP (UASB + WSP)	166.00	4.10	205.00	8.47	0.23	6.10	14.80	377.40	1927.16
4	Up Flow - Anaerobic Sludge Blanket Reactor+ ASP (UASB + ASP)	333.00	0.51	25.50	18.41	3.67	6.10	28.18	718.59	2216.13
5	Sequence Batch Reactor (SBR)	509.00	0.50	25.00	18.41	4.49	13.12	36.02	918.51	3149.16
6	Extended Aeration Process (EAP)	440.00	0.52	26.00	18.41	5.42	13.12	36.95	942.23	2888.82
7	Moving Bed Bio Reactor (MBBR)	276.00	0.43	21.50	18.41	6.53	13.12	38.06	970.53	2213.26
8	Anaerobic Pond + Aerated Facultative Pond (AP+AFP)	151.00	4.04	202.00	5.88	0.00	5.00	10.88	277.44	1752.01
9	Up Flow - Anaerobic Sludge Blanket Reactor+ Aerated Faculative Pond (UASBR + AFP)	175.00	3.38	169.00	8.47	0.23	6.10	14.80	377.40	1814.37
10	Bio Tower (Tricking Filter)	160.00	0.43	21.50	18.41	7.84	13.12	39.37	1003.83	1762.00

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Table 3.12 LCCA for 2 MLD plants with Rs. 50 lakhs per acre land cost

Life Cycle Cost (LCC) - Parameters - 2.0 MLD STP Capacity

S. No	Technologies	Capital Cost (Rs. In Lakhs)	Cost (Rs. In Lakhs)							
			Land		O&M Cost					LCC at the end of 15 Years
			Required (In Acres)	Cost@50Lakhs per Acre	Manpower	Power	Others	Total		
								Per Annum	15 Years (@10% escalation per annum)	
1	Waste Stabilization Ponds (WSP)	203.00	8.78	439.00	5.88	0.33	5.28	11.49	293.00	2974.79
2	Anaerobic Bafled Reactor (ABR) + Constructed Wet Lands (CWL)	455.00	7	350.00	5.88	0.33	5.28	11.49	293.00	3655.68
3	Up Flow - Anaerobic Sludge Blanket Reactor+ WSP (UASB + WSP)	234.00	7.90	395.00	8.47	7.34	10.18	25.99	662.75	3290.23
4	Up Flow - Anaerobic Sludge Blanket Reactor+ ASP (UASB + ASP)	431.00	0.64	32.00	18.41	7.34	10.18	35.93	916.22	2850.28
5	Sequence Batch Reactor (SBR)	696.00	0.55	27.50	18.41	8.98	24.20	51.59	1315.55	4337.78
6	Extended Aeration Process (EAP)	527.00	0.69	34.50	18.41	10.84	24.20	53.45	1362.98	3708.50
7	Moving Bed Bio Reactor (MBBR)	333.00	0.56	28.00	18.41	13.06	24.20	55.67	1419.59	2927.57
8	Anaerobic Pond + Aerated Facultative Pond (AP+AFP)	241.00	6.96	348.00	5.88	0.00	5.28	11.16	284.58	2018.14
9	Up Flow - Anaerobic Sludge Blanket Reactor+ Aerated Faculative Pond (UASBR + AFP)	239.00	6.58	329.00	8.47	7.34	10.18	25.99	662.75	2348.26
10	Bio Tower (Tricking Filter)	285.00	0.5	25.00	18.41	13.06	24.20	55.67	1419.59	2714.53

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Table 3. 13 LCCA for 5 MLD plants with Rs. 50 lakhs per acre land cost

Life Cycle Cost (LCC) - Parameters - 5.0 MLD STP Capacity

S. No	Technologies	Capital Cost (Rs. In Lakhs)	Cost (Rs. In Lakhs)							
			Land		O&M Cost					LCC at the end of 15 Years
			Required (In Acres)	Cost@50Lakhs per Acre	Manpower	Power	Others	Total		
								Per Annum	15 Years (@10% escalation per annum)	
1	Waste Stabilization Ponds (WSP)	384.00	20.25	1012.50	5.88	0.83	6.10	12.81	326.66	6160.18
2	Anaerobic Bafled Reactor (ABR) + Constructed Wet Lands (CWL)	1110.00	16.96	848.00	5.88	0.83	6.10	12.81	326.66	8505.71
3	Up Flow - Anaerobic Sludge Blanket Reactor+ WSP (UASB + WSP)	391.00	18.20	910.00	8.47	18.35	22.38	49.20	1254.60	6689.20
4	Up Flow - Anaerobic Sludge Blanket Reactor+ ASP (UASB + ASP)	659.00	0.83	41.50	18.41	18.35	22.38	59.14	1508.07	4434.23
5	Sequence Batch Reactor (SBR)	1163.00	0.86	43.00	18.41	22.44	57.43	98.28	2506.14	7543.90
6	Extended Aeration Process (EAP)	861.00	1.13	56.50	18.41	27.11	57.43	102.95	2625.23	6457.85
7	Moving Bed Bio Reactor (MBBR)	552.00	0.8	40.00	18.41	32.66	57.43	108.50	2766.75	5239.68
8	Anaerobic Pond + Aerated Facultative Pond (AP+AFP)	503.00	16.09	804.50	5.88	0.00	6.10	11.98	305.49	4086.94
9	Up Flow - Anaerobic Sludge Blanket Reactor+ Aerated Faculative Pond (UASBR + AFP)	444.00	15.30	765.00	8.47	18.35	22.38	49.20	1254.60	4707.10
10	Bio Tower (Tricking Filter)	488.00	0.76	38.00	18.41	32.66	57.43	108.50	2766.75	4963.98

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Table 3.14 LCCA for 1 MLD plants with Rs. 75 lakhs per acre land cost

Life Cycle Cost (LCC) - Parameters - 1.0 MLD STP Capacity

S. No	Technologies	Capital Cost (Rs. In Lakhs)	Cost (Rs. In Lakhs)							
			Land		O&M Cost					LCC at the end of 15 Years
			Required (In Acres)	Cost@75Lakhs per Acre	Manpower	Power	Others	Total		
								Per Annum	15 Years (@10% escalation per annum)	
1	Waste Stabilization Ponds (WSP)	136.00	5.04	378.00	5.88	0.17	5.00	11.05	281.78	2428.88
2	Anaerobic Bafled Reactor (ABR) + Constructed Wet Lands (CWL)	248.00	3.7	277.50	5.88	0.17	5.00	11.05	281.78	2476.92
3	Up Flow - Anaerobic Sludge Blanket Reactor+ WSP (UASB + WSP)	166.00	4.10	307.50	8.47	0.23	6.10	14.80	377.40	2355.33
4	Up Flow - Anaerobic Sludge Blanket Reactor+ ASP (UASB + ASP)	333.00	0.51	38.25	18.41	3.67	6.10	28.18	718.59	2269.39
5	Sequence Batch Reactor (SBR)	509.00	0.50	37.50	18.41	4.49	13.12	36.02	918.51	3201.38
6	Extended Aeration Process (EAP)	440.00	0.52	39.00	18.41	5.42	13.12	36.95	942.23	2943.13
7	Moving Bed Bio Reactor (MBBR)	276.00	0.43	32.25	18.41	6.53	13.12	38.06	970.53	2258.17
8	Anaerobic Pond + Aerated Facultative Pond (AP+AFP)	151.00	4.04	303.00	5.88	0.00	5.00	10.88	277.44	2173.91
9	Up Flow - Anaerobic Sludge Blanket Reactor+ Aerated Faculative Pond (UASBR + AFP)	175.00	3.38	253.50	8.47	0.23	6.10	14.80	377.40	2167.35
10	Bio Tower (Tricking Filter)	160.00	0.43	32.25	18.41	7.84	13.12	39.37	1003.83	1806.91

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Table 3. 15 LCCA for 2 MLD plants with Rs. 75 lakhs per acre land cost

Life Cycle Cost (LCC) - Parameters - 2.0 MLD STP Capacity

S. No	Technologies	Capital Cost (Rs. In Lakhs)	Cost (Rs. In Lakhs)							
			Land		O&M Cost					LCC at the end of 15 Years
			Required (In Acres)	Cost@75Lakhs per Acre	Manpower	Power	Others	Total		
								Per Annum	15 Years (@10% escalation per annum)	
1	Waste Stabilization Ponds (WSP)	203.00	8.78	658.50	5.88	0.33	5.28	11.49	293.00	3891.69
2	Anaerobic Bafled Reactor (ABR) + Constructed Wet Lands (CWL)	455.00	7	525.00	5.88	0.33	5.28	11.49	293.00	4386.70
3	Up Flow - Anaerobic Sludge Blanket Reactor+ WSP (UASB + WSP)	234.00	7.90	592.50	8.47	7.34	10.18	25.99	662.75	4115.24
4	Up Flow - Anaerobic Sludge Blanket Reactor+ ASP (UASB + ASP)	431.00	0.64	48.00	18.41	7.34	10.18	35.93	916.22	2917.12
5	Sequence Batch Reactor (SBR)	696.00	0.55	41.25	18.41	8.98	24.20	51.59	1315.55	4395.22
6	Extended Aeration Process (EAP)	527.00	0.69	51.75	18.41	10.84	24.20	53.45	1362.98	3780.56
7	Moving Bed Bio Reactor (MBBR)	333.00	0.56	42.00	18.41	13.06	24.20	55.67	1419.59	2986.05
8	Anaerobic Pond + Aerated Facultative Pond (AP+AFP)	241.00	6.96	522.00	5.88	0.00	5.28	11.16	284.58	3471.82
9	Up Flow - Anaerobic Sludge Blanket Reactor+ Aerated Faculative Pond (UASBR + AFP)	239.00	6.58	493.50	8.47	7.34	10.18	25.99	662.75	3722.58
10	Bio Tower (Tricking Filter)	285.00	0.5	37.50	18.41	13.06	24.20	55.67	1419.59	2766.75

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Table 3. 16 LCCA for 5 MLD plants with Rs. 75 lakhs per acre

Life Cycle Cost (LCC) - Parameters - 5.0 MLD STP Capacity

S. No	Technologies	Capital Cost (Rs. In Lakhs)	Cost (Rs. In Lakhs)							
			Land		O&M Cost					LCC at the end of 15 Years
			Required (In Acres)	Cost@75Lakhs per Acre	Manpower	Power	Others	Total		
								Per Annum	15 Years (@10% escalation per annum)	
1	Waste Stabilization Ponds (WSP)	384.00	20.25	1518.75	5.88	0.83	6.10	12.81	326.66	8274.91
2	Anaerobic Bafled Reactor (ABR) + Constructed Wet Lands (CWL)	1110.00	16.96	1272.00	5.88	0.83	6.10	12.81	326.66	10276.86
3	Up Flow - Anaerobic Sludge Blanket Reactor+ WSP (UASB + WSP)	391.00	18.20	1365.00	8.47	18.35	22.38	49.20	1254.60	8589.85
4	Up Flow - Anaerobic Sludge Blanket Reactor+ ASP (UASB + ASP)	659.00	0.83	62.25	18.41	18.35	22.38	59.14	1508.07	4520.91
5	Sequence Batch Reactor (SBR)	1163.00	0.86	64.50	18.41	22.44	57.43	98.28	2506.14	7633.71
6	Extended Aeration Process (EAP)	861.00	1.13	84.75	18.41	27.11	57.43	102.95	2625.23	6575.86
7	Moving Bed Bio Reactor (MBBR)	552.00	0.8	60.00	18.41	32.66	57.43	108.50	2766.75	5323.23
8	Anaerobic Pond + Aerated Facultative Pond (AP+AFP)	503.00	16.09	1206.75	5.88	0.00	6.10	11.98	305.49	7447.54
9	Up Flow - Anaerobic Sludge Blanket Reactor+ Aerated Faculative Pond (UASBR + AFP)	444.00	15.30	1147.50	8.47	18.35	22.38	49.20	1254.60	7902.69
10	Bio Tower (Tricking Filter)	488.00	0.76	57.00	18.41	32.66	57.43	108.50	2766.75	5043.35

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Table 3. 17 LCCA for 1 MLD plants with Rs. 100 lakhs per acre land cost

Life Cycle Cost (LCC) - Parameters - 1.0 MLD STP Capacity

S. No	Technologies	Capital Cost (Rs. In Lakhs)	Cost (Rs. In Lakhs)							
			Land		O&M Cost					LCC at the end of 15 Years
			Required (In Acres)	Cost@100Lakhs per Acre	Manpower	Power	Others	Total		
								Per Annum	15 Years (@10% escalation per annum)	
1	Waste Stabilization Ponds (WSP)	136.00	5.04	504.00	5.88	0.17	5.00	11.05	281.78	2955.21
2	Anaerobic Bafled Reactor (ABR) + Constructed Wet Lands (CWL)	248.00	3.7	370.00	5.88	0.17	5.00	11.05	281.78	2863.31
3	Up Flow - Anaerobic Sludge Blanket Reactor+ WSP (UASB + WSP)	166.00	4.10	410.00	8.47	0.23	6.10	14.80	377.40	2783.49
4	Up Flow - Anaerobic Sludge Blanket Reactor+ ASP (UASB + ASP)	333.00	0.51	51.00	18.41	3.67	6.10	28.18	718.59	2322.65
5	Sequence Batch Reactor (SBR)	509.00	0.50	50.00	18.41	4.49	13.12	36.02	918.51	3253.59
6	Extended Aeration Process (EAP)	440.00	0.52	52.00	18.41	5.42	13.12	36.95	942.23	2997.43
7	Moving Bed Bio Reactor (MBBR)	276.00	0.43	43.00	18.41	6.53	13.12	38.06	970.53	2303.07
8	Anaerobic Pond + Aerated Facultative Pond (AP+AFP)	151.00	4.04	404.00	5.88	0.00	5.00	10.88	277.44	2595.81
9	Up Flow - Anaerobic Sludge Blanket Reactor+ Aerated Faculative Pond (UASBR + AFP)	175.00	3.38	338.00	8.47	0.23	6.10	14.80	377.40	2520.33
10	Bio Tower (Tricking Filter)	160.00	0.43	43.00	18.41	7.84	13.12	39.37	1003.83	1851.81

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Table 3. 18 LCCA for 2 MLD plants with Rs. 100 lakhs per acre land cost

Life Cycle Cost (LCC) - Parameters - 2.0 MLD STP Capacity

S. No	Technologies	Capital Cost (Rs. In Lakhs)	Cost (Rs. In Lakhs)							
			Land		O&M Cost					LCC at the end of 15 Years
			Required (In Acres)	Cost@100Lakhs per Acre	Manpower	Power	Others	Total		
								Per Annum	15 Years (@10% escalation per annum)	
1	Waste Stabilization Ponds (WSP)	203.00	8.78	878.00	5.88	0.33	5.28	11.49	293.00	4808.60
2	Anaerobic Bafled Reactor (ABR) + Constructed Wet Lands (CWL)	455.00	7	700.00	5.88	0.33	5.28	11.49	293.00	5117.72
3	Up Flow - Anaerobic Sludge Blanket Reactor+ WSP (UASB + WSP)	234.00	7.90	790.00	8.47	7.34	10.18	25.99	662.75	4940.25
4	Up Flow - Anaerobic Sludge Blanket Reactor+ ASP (UASB + ASP)	431.00	0.64	64.00	18.41	7.34	10.18	35.93	916.22	2983.95
5	Sequence Batch Reactor (SBR)	696.00	0.55	55.00	18.41	8.98	24.20	51.59	1315.55	4452.66
6	Extended Aeration Process (EAP)	527.00	0.69	69.00	18.41	10.84	24.20	53.45	1362.98	3852.61
7	Moving Bed Bio Reactor (MBBR)	333.00	0.56	56.00	18.41	13.06	24.20	55.67	1419.59	3044.53
8	Anaerobic Pond + Aerated Facultative Pond (AP+AFP)	241.00	6.96	696.00	5.88	0.00	5.28	11.16	284.58	4198.66
9	Up Flow - Anaerobic Sludge Blanket Reactor+ Aerated Faculative Pond (UASBR + AFP)	239.00	6.58	658.00	8.47	7.34	10.18	25.99	662.75	4409.74
10	Bio Tower (Tricking Filter)	285.00	0.5	50.00	18.41	13.06	24.20	55.67	1419.59	2818.96

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Table 3. 19 LCCA for 5 MLD plants with Rs. 100 lakhs per acre land cost

Life Cycle Cost (LCC) - Parameters - 5.0 MLD STP Capacity

S. No	Technologies	Capital Cost (Rs. In Lakhs)	Cost (Rs. In Lakhs)							
			Land		O&M Cost					LCC at the end of 15 Years
			Required (In Acres)	Cost@100Lakhs per Acre	Manpower	Power	Others	Total		
								Per Annum	15 Years (@10% escalation per annum)	
1	Waste Stabilization Ponds (WSP)	384.00	20.25	2025.00	5.88	0.83	6.10	12.81	326.66	10389.65
2	Anaerobic Bafled Reactor (ABR) + Constructed Wet Lands (CWL)	1110.00	16.96	1696.00	5.88	0.83	6.10	12.81	326.66	12048.01
3	Up Flow - Anaerobic Sludge Blanket Reactor+ WSP (UASB + WSP)	391.00	18.20	1820.00	8.47	18.35	22.38	49.20	1254.60	10490.50
4	Up Flow - Anaerobic Sludge Blanket Reactor+ ASP (UASB + ASP)	659.00	0.83	83.00	18.41	18.35	22.38	59.14	1508.07	4607.59
5	Sequence Batch Reactor (SBR)	1163.00	0.86	86.00	18.41	22.44	57.43	98.28	2506.14	7723.52
6	Extended Aeration Process (EAP)	861.00	1.13	113.00	18.41	27.11	57.43	102.95	2625.23	6693.86
7	Moving Bed Bio Reactor (MBBR)	552.00	0.8	80.00	18.41	32.66	57.43	108.50	2766.75	5406.77
8	Anaerobic Pond + Aerated Facultative Pond (AP+AFP)	503.00	16.09	1609.00	5.88	0.00	6.10	11.98	305.49	9127.84
9	Up Flow - Anaerobic Sludge Blanket Reactor+ Aerated Faculative Pond (UASBR + AFP)	444.00	15.30	1530.00	8.47	18.35	22.38	49.20	1254.60	9500.49
10	Bio Tower (Tricking Filter)	488.00	0.76	76.00	18.41	32.66	57.43	108.50	2766.75	5122.72

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STP Project Life Cycle Cost

Sl. No.	Technologies	Zero Land Cost (Rs. In Lakhs)			10 Lakhs Land Cost (Rs. In Lakhs)			25 Lakhs Land Cost (Rs. In Lakhs)			50 Lakhs Land Cost (Rs. In Lakhs)			75 Lakhs Land Cost (Rs. In Lakhs)			100 Lakhs Land Cost (Rs. In Lakhs)		
		1 MLD	2 MLD	5 MLD	1 MLD	2 MLD	5 MLD	1 MLD	2 MLD	5 MLD	1 MLD	2 MLD	5 MLD	1 MLD	2 MLD	5 MLD	1 MLD	2 MLD	5 MLD
1	Waste Stabilization Ponds (WSP)	850	1141	1931	1060	1508	2777	1376	2058	4045	1903	2975	6160	2429	3892	8275	2955	4809	10390
2	Anaerobic Bafled Reactor (ABR) + Constructed Wet Lands (CWL)	1318	2194	4963	1472	2486	5672	1704	2925	6735	2091	3656	8506	2477	4387	10277	2863	5118	12048
3	Up Flow - Anaerobic Sludge Blanket Reactor+ WSP (UASB + WSP)	1071	1640	2888	1330	1970	3648	1499	2465	4789	1927	3290	6689	2355	4115	8590	2783	4940	10490
4	Up Flow - Anaerobic Sludge Blanket Reactor + ASP (UASB + ASP)	2110	2717	4261	2131	2743	4296	2163	2783	4348	2216	2850	4434	2269	2917	4521	2323	2984	4608
5	Sequence Batch Reactor (SBR)	3045	4223	7364	3066	4246	7400	3097	4280	7454	3149	4338	7544	3201	4395	7634	3254	4453	7724
6	Extended Aeration Process (EAP)	2780	3564	6222	2802	3593	6269	2835	3636	6340	2889	3708	6458	2943	3781	6576	2997	3853	6694
7	Moving Bed Bio Reactor (MBBR)	2123	2811	5073	2141	2834	5106	2168	2869	5156	2213	2928	5240	2258	2986	5323	2303	3045	5407
8	Anaerobic Pond + Aerated Facultative Pond (AP+AFP)	913	1300	2427	1081	1590	3099	1330	2018	4086	1752	2744	5767	2174	3471	7447	2596	4198	9127
9	Upflow Anaerobic Sludge Blanket Reactor + Aerated Faculative Pond (UASBR + AFP)	1108	1661	3109	1337	1935	3748	1461	2348	4707	1814	3035	6304	2167	3722	7902	2520	4409	9500
10	Bio Tower (Tricking Filter)	1672	2610	4805	1690	2631	4837	1717	2662	4885	1762	2715	4964	1807	2767	5043	1852	2819	5123

Technology		Capacity	Land cost (Rs. / acre)
Anaerobic Pond + Aerated Facultative Pond		1 MLD, 2 MLD, 5 MLD	Rs. 0, 10, 25, 50
Bio-Tower (Trickling Filter)		1 MLD, 2 MLD	Rs. 75 lakhs and Rs. 100 lakhs
UASB Reactor + Activated Sludge Process		5 MLD	Rs. 75 lakhs and Rs. 100 lakhs
NOTE	The technology with lowest LCC for each combination of capacity and land cost is highlighted in green. Alternate technology very close to the lowest LCC technologoy is highlighted in blue. Technology with the highest LCC is heighted in Red.		
Conclusion	For all STP capacities up to Rs. Rs. 50 lakhs / acre land cost, (UASBR + AFP) has emerged as the least LCC technology and SBR as the highest LCC technology. For land cost of Rs. 75 and 100 lakhs/acre Bio-Tower is lowest LCC option up to 2 MLD and UASBR + ASP for 5 MLD.		

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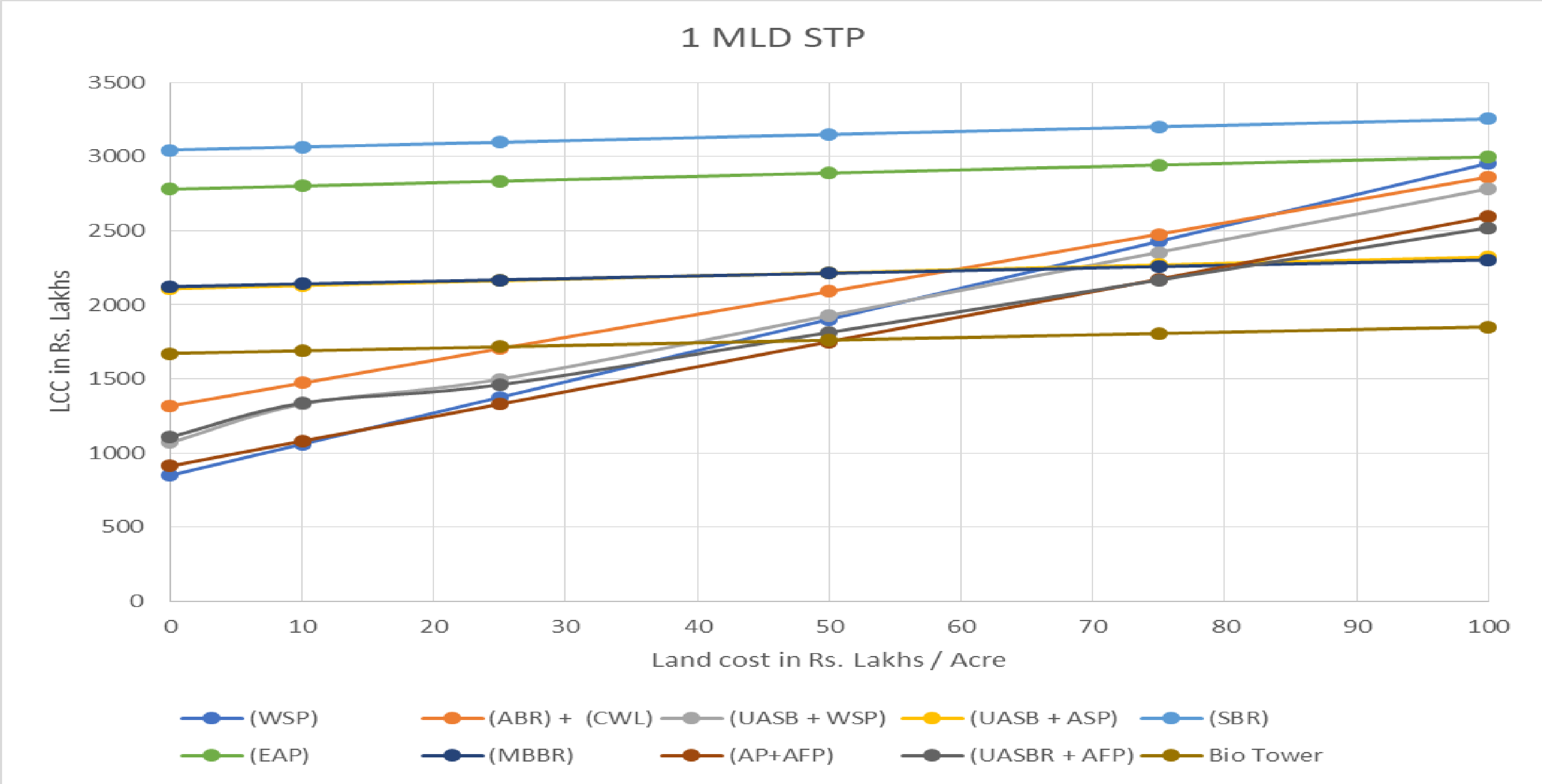
Table 3. 20 Summary of LCCA of 10 Technologies – 3 Capacities – 6 Land costs

Figure 3. 2 Plot of LCC for 1 MLD plants of the ten technologies chosen for study



NOTE

The tables contain discrete data for 1, 2 and 5 MLD STPs. Graphical presentation of the same data converts it to a continuous form so that decisions can be made even for intermediate STP capacities. However, extrapolation is not advised.

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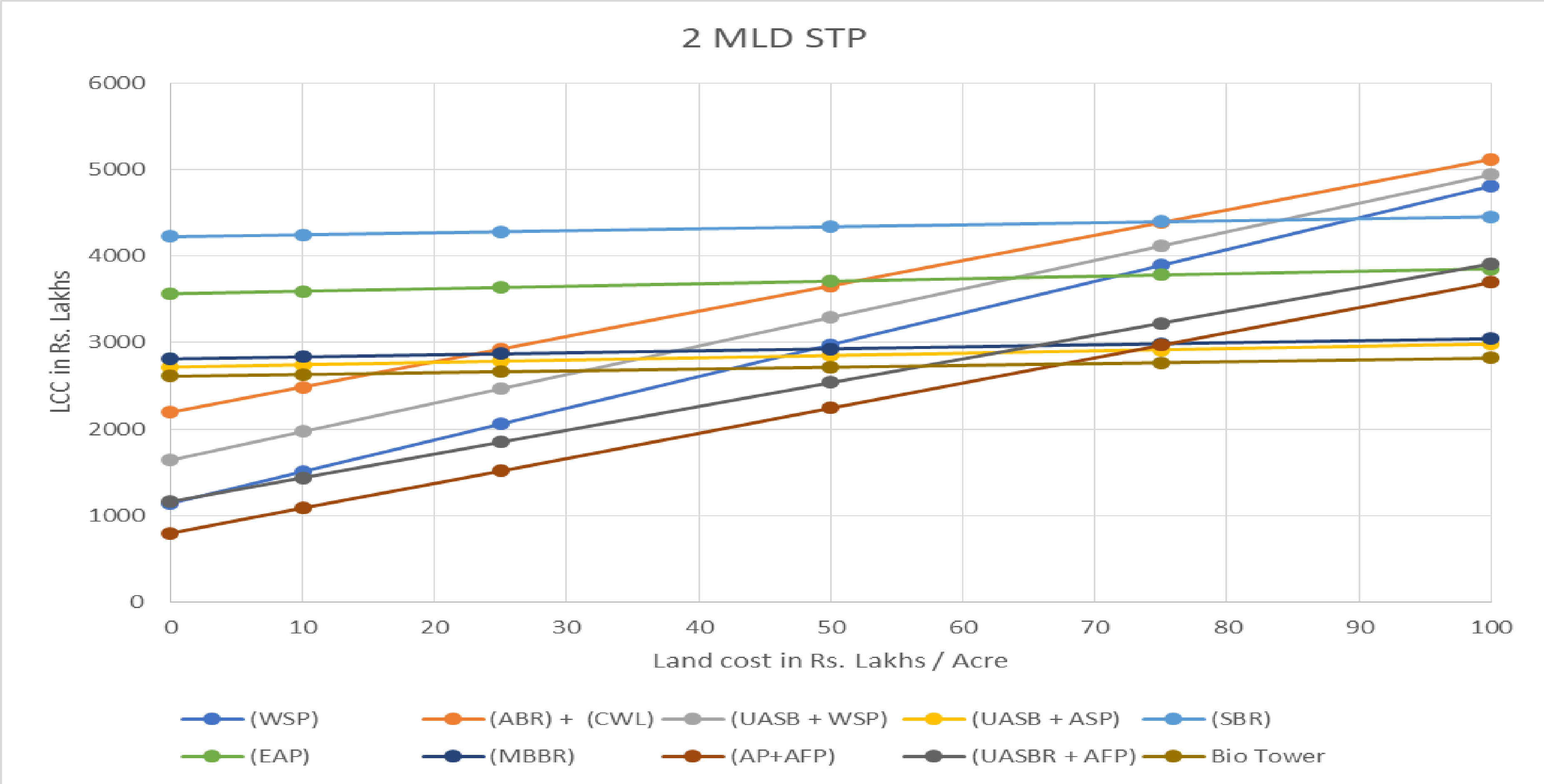
Figure 3. 3 Plot of LCC for 2 MLD plants of the ten technologies chosen for study



NOTE

The tables contain discrete data for 1, 2 and 5 MLD STPs. Graphical presentation of the same data converts it to a continuous form so that decisions can be made even for intermediate STP capacities. However, extrapolation is not advised.

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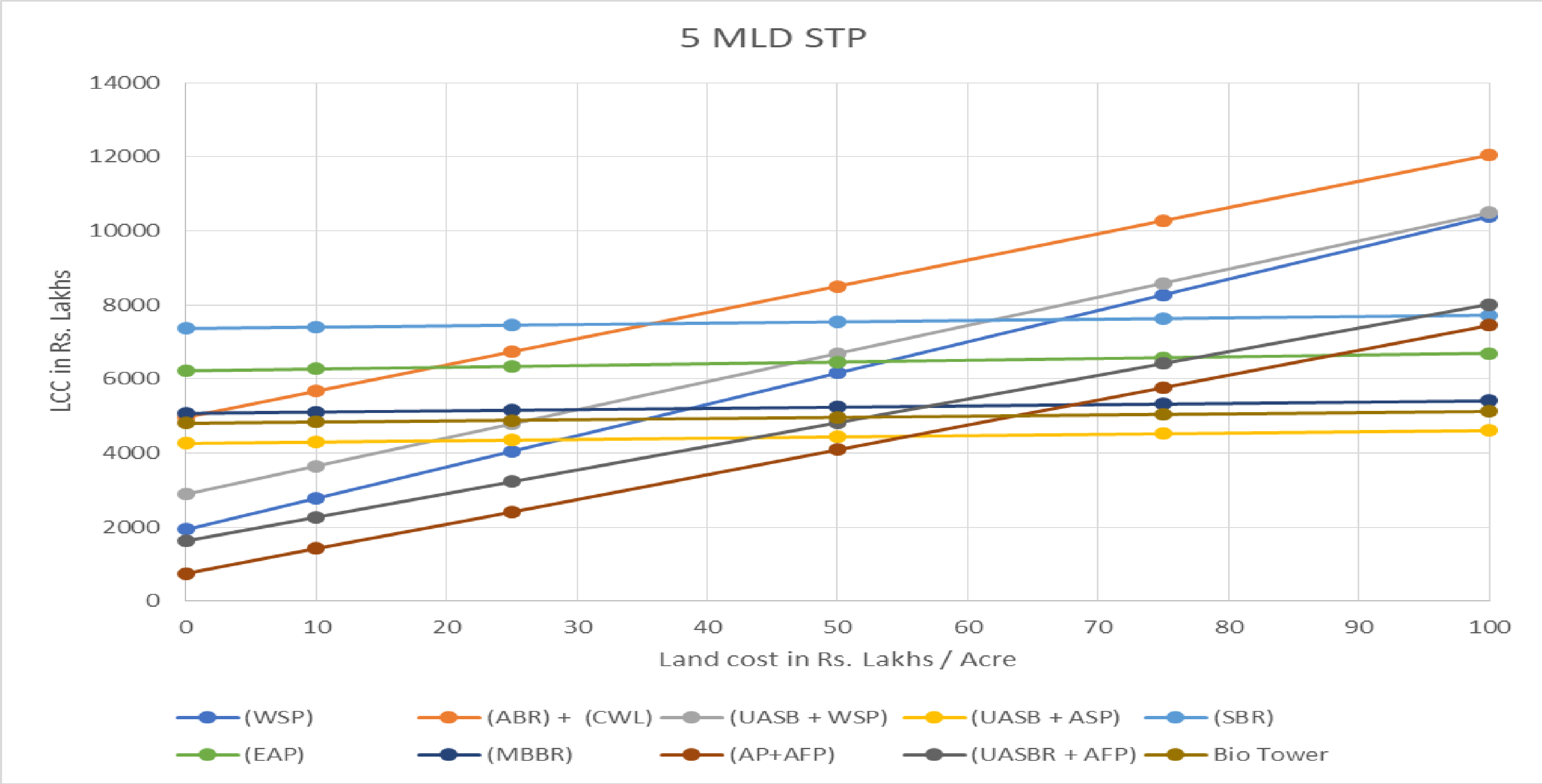
Figure 3. 4 Plot of LCC for 5 MLD plants of the ten technologies chosen for study



NOTE

The tables contain discrete data for 1, 2 and 5 MLD STPs. Graphical presentation of the same data converts it to a continuous form so that decisions can be made even for intermediate STP capacities. However, extrapolation is not advised.

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CHAPTER – 4

USED / GREY WATER MANAGEMENT IN CLASS V AND VI TOWNS

4.1 On-Site Sanitation

On-site sanitation is covered under Chapter – 9 of the CPHEEO's 'Manual on Sewerage and Sewage Treatment Systems - 2013'. An exhaustive coverage is presented on various options for toilets, their design, materials and construction. This is followed by the design and construction of septic tanks of various types. Section 9.3.4.4 deals with secondary treatment and disposal of septic tank effluent.

The septic tank effluent contains not only dissolved organic content but also pathogenic organisms and hence, needs to be treated before its final, safe disposal. Depending upon the situation — the size, treatment objective, resources available etc., the extent and type of secondary treatment facility can vary from the most conventional land disposal methods like soak pits or dispersion trenches to additional secondary biological treatment systems.

The most conventional land disposal method consist of soak pits or dispersion trenches which are designed to achieve seepage into the surrounding soil. The soak pit size depends upon the porosity and percolation characteristics of the soil.

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4.2 Soak Pits for Grey Water Management

A brief description of Soak pits is presented in section 9.3.4.5 of the CPHEEO's manual. They are cheap and easy to construct and use. The land area required is small and the capital cost and operating cost is low. Soak pits are said to be capable of recharging ground water.

SOAK PITS

1. Minimum plan area of rectangular soak pit is 1 x 1 m and minimum diameter of a circular pit is 1 m.
2. Depth of pit below inlet pipe is 1 m.
3. The pit should be covered and the top raised above the adjacent ground in order to avoid flooding during rainfall.

The following parameters are of interest in locating and designing of soak pits.

i. The soil should be porous with soak percolation rate below 25 minutes per cm.

ii. For percolation rates between 12 and 25 minutes / cm, dispersion trenches should be preferred.

iii. The depth of water table must be 2 m or more from the ground level.

iv. Distance between the soak pits and nearby water bodies / water sources should be minimum of 30 m.

v. The soak pit should not be closer than 7 m from a dwelling unit in order to avoid any corrosive effect due to tank gases vented into atmosphere.

4.3 Design Reference followed

The publication titled 'Technological Approaches and Design Brief on Liquid Waste Management for Rural Areas of Punjab' has been brought out by WASH Institute for the Department of Water Supply and Sanitation, Punjab under Liquid Waste Management component of Swachh Bharat Mission – Grameen.

The 'type designs' presented in the above mentioned publication pertain to management of grey water at household level and community level. It is to be stated that these 'type designs'

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may be used for class V (population 5000 – 9999) and class VI (population < 5000) towns under SBM Urban 2.0. The estimated sewage quantity for class V towns is 0.5 MLD to 1.0 MLD and for class VI towns it is less than 0.5 MLD.

The following assumptions are made in the document in order to arrive at the size of the pits:

i. **Percolation rate of soil:** The IS : 2470 (Part 2) : 1985 (Reaffirmed 2001) provides detailed information regarding the Maximum rate of effluent application, Q (l/m2/day) to standard percolation rate, t (minutes) based on the formula:

$$Q = \frac{204}{\sqrt{t}} \quad Equation\ 4.1$$

Owing to the difficulty in obtaining data at the site, HRT is used as a key design parameter. Table 2 of IS : 2470 (Part 2) presents the numerical output of equation 4.1.

ii. **Hydraulic retention time (HRT)**: Based on a few reference projects implemented in Maharashtra (guided by World Bank) and their performance, it is recommended to assume 2 days of HRT (48 hours) for sizing the pits.

iii. **Filter media:** As per the ground practice, soak pits without filter media are used for high discharge of wastewater. In the present designs, soak pits for 3 to 10 KLD are designed without filter media.

Table 2 of IS : 2470 (Part 2) : 1985

Percolation Rate (Minutes)	Maximum Rate of Effluent Application (l /m^2/day)
1 or less	204
2	143
3	118
4	102
5	90
10	65
15	52
30	37
45	33
60	26

4.3.1 Types of Soak Pits

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Based upon the provision of porous lining and presence or absence of media, the soak pits are classified into four categories as described alongside in Figure 4.1.

Each of these pits are used in different situations depending on the quantity and concentration of wastewater to be discharged into the soak pits.

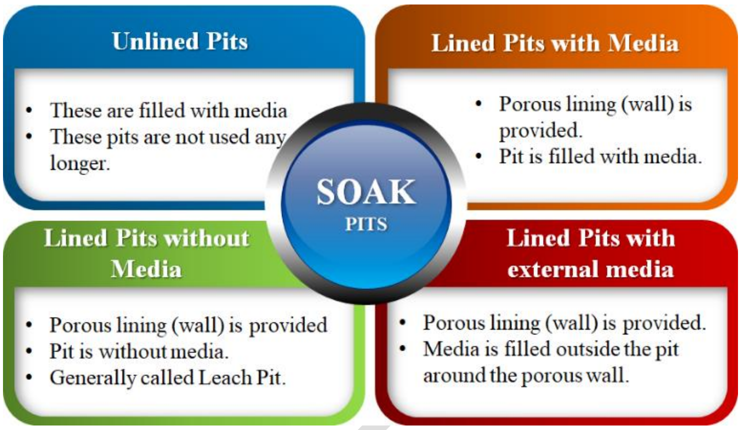

Figure 4.1 Classification of soak pits

In the reference document of Punjab, the following classification is used for 'type design' of the soak pits.

i. Type 'B' nomenclature is used for media filled pits

ii. Type 'C' nomenclature refers to pits without media

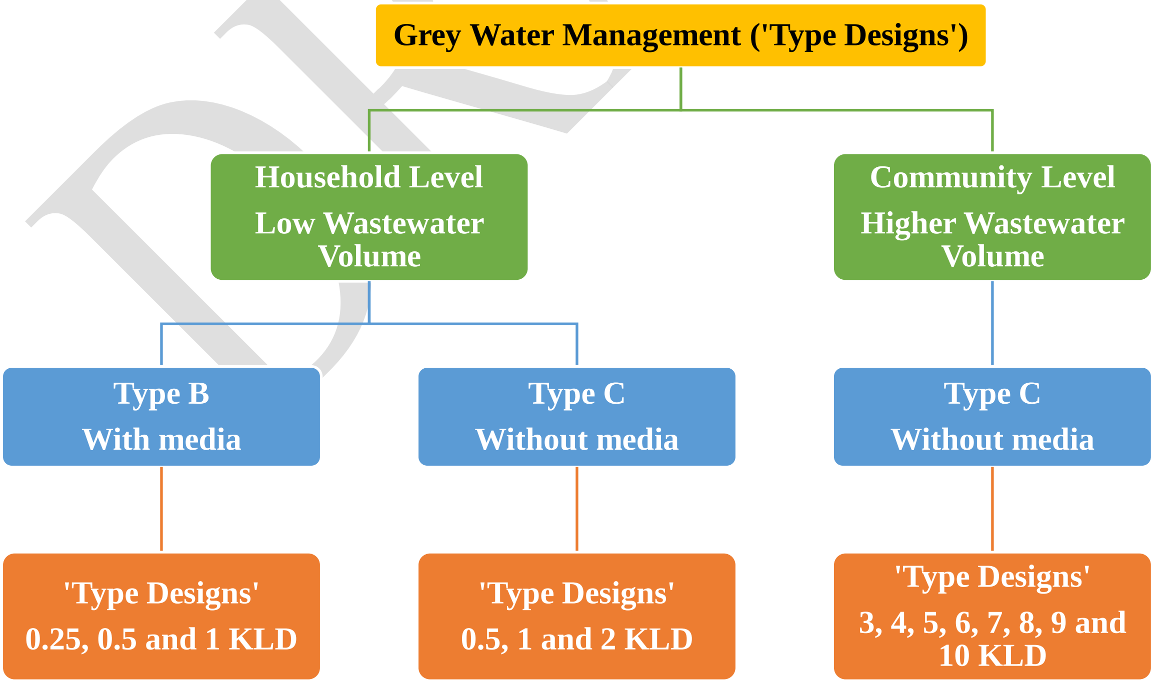

Figure 4. 1 Details of Type B and Type C soak pits for households and community

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4.3.1.1 Household Level 'Type Designs'

The soak pit is one of the most used techniques to discharge the following liquid waste at household level:

a) Blackwater effluent from household septic tanks.
b) Greywater from the households with or without pre-treatment.
c) Mixed wastewater (grey water and effluent from septic tanks) discharged into the drain with pre-treatment.

The soak pit, which is lined with a porous wall to prevent it from collapsing, receives wastewater and allows it to slowly drain away through the holes / openings in the porous wall, to get absorbed into the surrounding soil. The grey water consisting of septic tank effluent and the waste water from household activities contains solid particles such as sand/silt, organic materials, and other smaller solid particles due to different household activities. These solid particles may either settle down in the soak pit or get filtered out by the soil matrix. However, the unsettled or suspended solid particles in raw grey water and the digested sludge particles in the soak pit may enter the surrounding soil surface along with water and may slowly reduce the percolation capacity of the soil and lead to the clogging of soil pores at a later stage. In order to avoid clogging as well as to ensure effective percolation of water into the surrounding soil surface, it is recommended to have an appropriate pre-treatment system prior to the entry of wastewater into the soak pits.

Provision of a settling tank prior to the soak pit is the simplest pre-treatment which will reduce the load of solids in the grey water and extend the lifespan of the soak pit. Periodic desludging of the septic tank will also reduce the load of the solids in the septic tank effluent. It is possible for a well sized / designed soak pit to last for 3 to 5 years without maintenance. Maintenance involves excavation, cleaning, desludging and either washing and reclaiming the material or replacing with new gravel, rock and sand based on the type of soak pits.

4.3.1.2 Sizing of Household Level 'Type Designs'

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As depicted in Figure 4.2, Type B and Type C soak pits are adopted at household level. It is assumed that the greywater is collected from the household through dedicated pipes and discharged into the soak pit. It is proposed to have the following components:

- Nahani trap and settling tank as pre-treatment system.
- Nahani trap to be installed at each of the greywater sources at the household level. This also acts as a screening system hence a separate screening system is not considered.
- Settling tank to arrest most of the suspended and floating particulate solids flowing with the wastewater.
- Type B or Type C soak pit.

For household level management of the greywater, 'type design' of 0.25 - 0.5, 1 and 2 KLD settling tank is presented in Table 4.1. The settling tank is followed by either Type B or Type C soak pit. As already mentioned, if the flow is higher, then Type C pit is recommended.

The 'type design' of Type B soak pit with media for 0.25, 0.5 and 1.0 KLD flow is presented in Table 4.2. Lastly, the 'type design' of Type C soak pit without media for 0.5, 1.0 and 2.0 KLD flow is presented in Table 4.3. The corresponding drawing are presented in Figures 4.3 to 4.7.

Table 4. 1 Sizing of the settling tank for household level Type B or Type C soak pits

Volume of greywater in KLD	**0.25 - 0.5**	**1**	**2**
Hydraulic Retention Time (HRT) in minutes	20	20	20
Effective length in m	0.7	0.9	1.0
Effective width in m	0.3	0.3	0.3
Effective Depth in m	0.4	0.5	0.5

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Table 4. 2 Sizing of Type B Soak Pit with media

Volume of greywater in KLD	**0.25**	**0.5**	**1**
Hydraulic Retention Time proposed in days	2	2	2
Clear Diameter in m	1.0	1.3	1.6
Effective Depth in m	1.3	1.6	2.1
Depth of filter media in m	1.15	1.45	1.95

Table 4. 3 Sizing of Type C Soak Pit without media

Volume of greywater in KLD	**0.5**	**1**	**2**
Hydraulic Retention Time proposed in days	2	2	2
Clear Diameter in m	1.0	1.3	1.7
Effective Depth in m	1.3	1.5	1.8

4.3.1.3 Sizing of Community Level 'Type Designs'

It is proposed that for management of grey water at the community level, only Type C Soak pit – without filter media will be used. The components for this approach are listed here by considering two types of outlets into the soak pit; the first type is an outlet from a cluster of households and the second one is outfall of a drain.

The grey water collected from a cluster of households at community level through a network of pipes or drains is discharged into the soak pit with the following arrangements:

- Nahani trap and settling tank as pre-treatment system.
- Nahani trap to be installed at each of the greywater sources at the household level. This also acts as a screening system hence a separate screening system is not considered.
- Settling tank to arrest most of the suspended and floating particulate solids flowing with the wastewater.

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- Type C Soak pit.

The grey water from the outfall of a drain is discharged into the soak pit with the following arrangements:

- Screening system at the end of the drain or at the entry of the I&D structure to remove the larger solid particles.
- I & D structure with overflow to divert the additional volume during the rainy season.
- Settling tank to arrest most of the suspended and floating particulate solids flowing with the wastewater.
- Type C Soak pit.

The I&D structure, screens and settling tank can be integrated into a single unit within the pre-treatment system. Further, it is recommended to have drain continuation beyond I & D structure in order to avoid back flow of greywater within the soak pit or drain as overflow system within the I & D system helps to divert the same.

For community level management of the greywater, 'type design' of 3 to 10 KLD settling tank is presented in Table 4.4. The sizing of pre-treatment tank of 3-5 and 6-10 KLD capacity, which follows the settling tank is presented in Table 4.5. As already mentioned, for community level soak pit, only Type C is recommended, which is located after the pre-treatment tank. The 'type design' of Type C soak pit without media for 3 - 10 KLD flow is presented in Table 4.6. The corresponding drawing are presented in Figures 4.8 to 4.12.

Table 4. 4 Sizing of the settling tank for Community level soak pit

Volume of greywater in KLD	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
HRT proposed in min	30	30	30	30	30	30	30	30
Effective length in m	1.0	1.2	1.2	1.2	1.2	1.4	1.4	1.5
Effective width in m	0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.5
Effective Depth in m	0.7	0.8	0.8	0.8	0.9	0.9	1.0	1.0

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Table 4. 5 Sizing of the pre-treatment tank for Community level soak pit

Volume of greywater in KLD	**3 – 5**	**6 - 10**
Hydraulic Retention Time proposed in min	15-20	15-20
Width of drain at the inlet in m	0.4	0.5
Width of the drain before the screen in m	0.55	0.65
Width of the drain after the screen in m	0.55	0.65
Maximum depth of the Settling tank near the overflow wall in m	0.5	0.5
Length of the settling tank in m	1.2	2.3

Table 4. 6 Sizing of the Type C Soak Pit for Community level disposal

Volume of greywater in KLD	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
HRT proposed in days	2	2	2	2	2	2	2	2
Clear Diameter in m	1.9	2.1	2.25	2.4	2.5	2.7	2.8	3
Effective Depth in m	2.1	2.3	2.5	2.7	2.8	2.9	2.9	3

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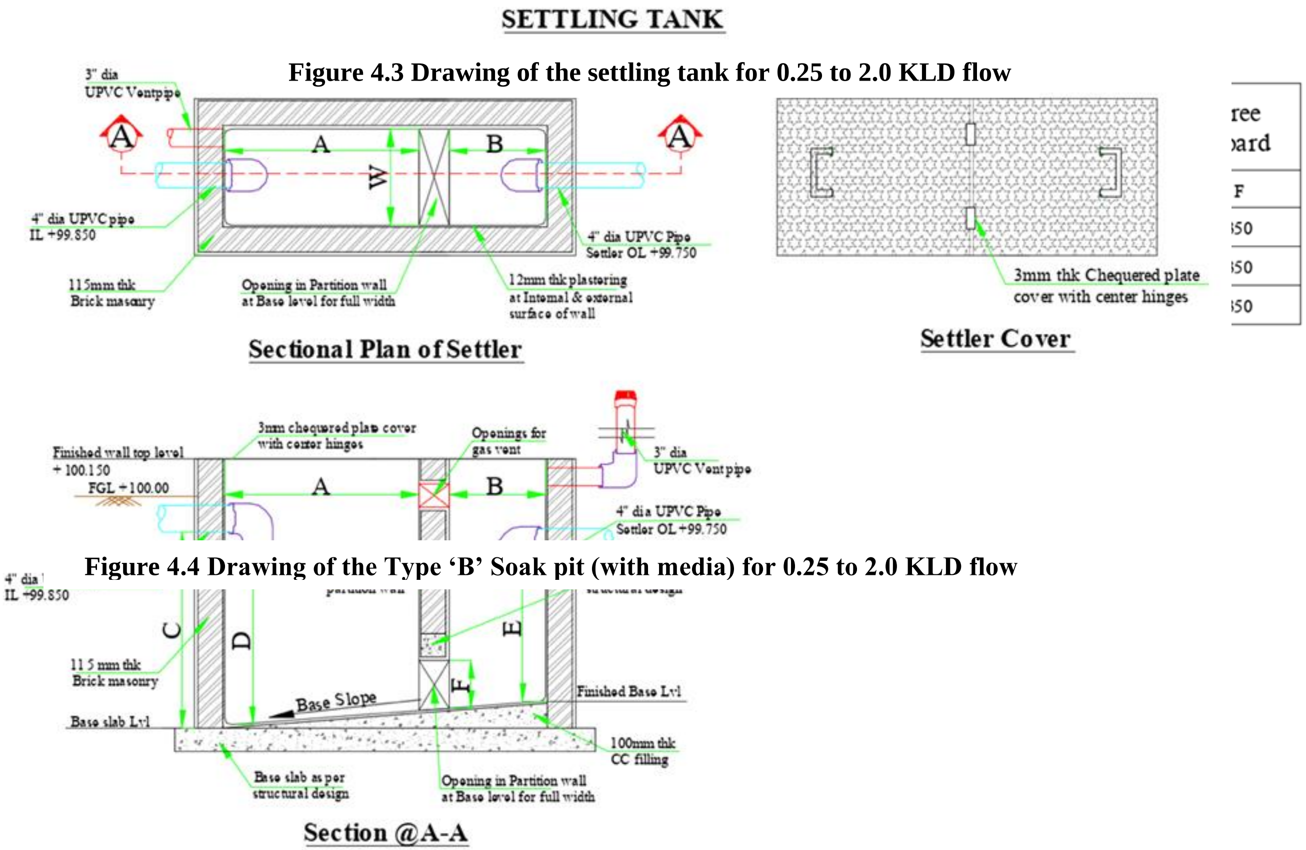
SOAK PIT TYPE B (WITH MEDIA) FOR HOUSEHOLD LEVEL DISPOSAL OF GREY WATER IN CLASS V AND CLASS VI TOWNS UNDER SBM U 2.0

SETTLING TANK

Figure 4.3 Drawing of the settling tank for 0.25 to 2.0 KLD flow

Figure 4.4 Drawing of the Type 'B' Soak pit (with media) for 0.25 to 2.0 KLD flow



Wastewater Volume	Length 1	Length 2	Inlet side Depth	Hydraulic Depth	Outlet side Depth	Bottom opening	Width
KLD	A	B	C	D	E	F	W
0.25-0.5	400	300	500	400	300	100	300
1	600	300	600	500	400	200	300
2	700	300	600	500	400	200	300

*All Dimensions in mm

The drawing is not to scale. Refer to the table for the dimensions

SETTLING TANK FOR HOUSEHOLD LEVEL DISPOSAL OF GREY WATER IN CLASS V AND CLASS VI TOWNS UNDER SBM URBAN 2.0

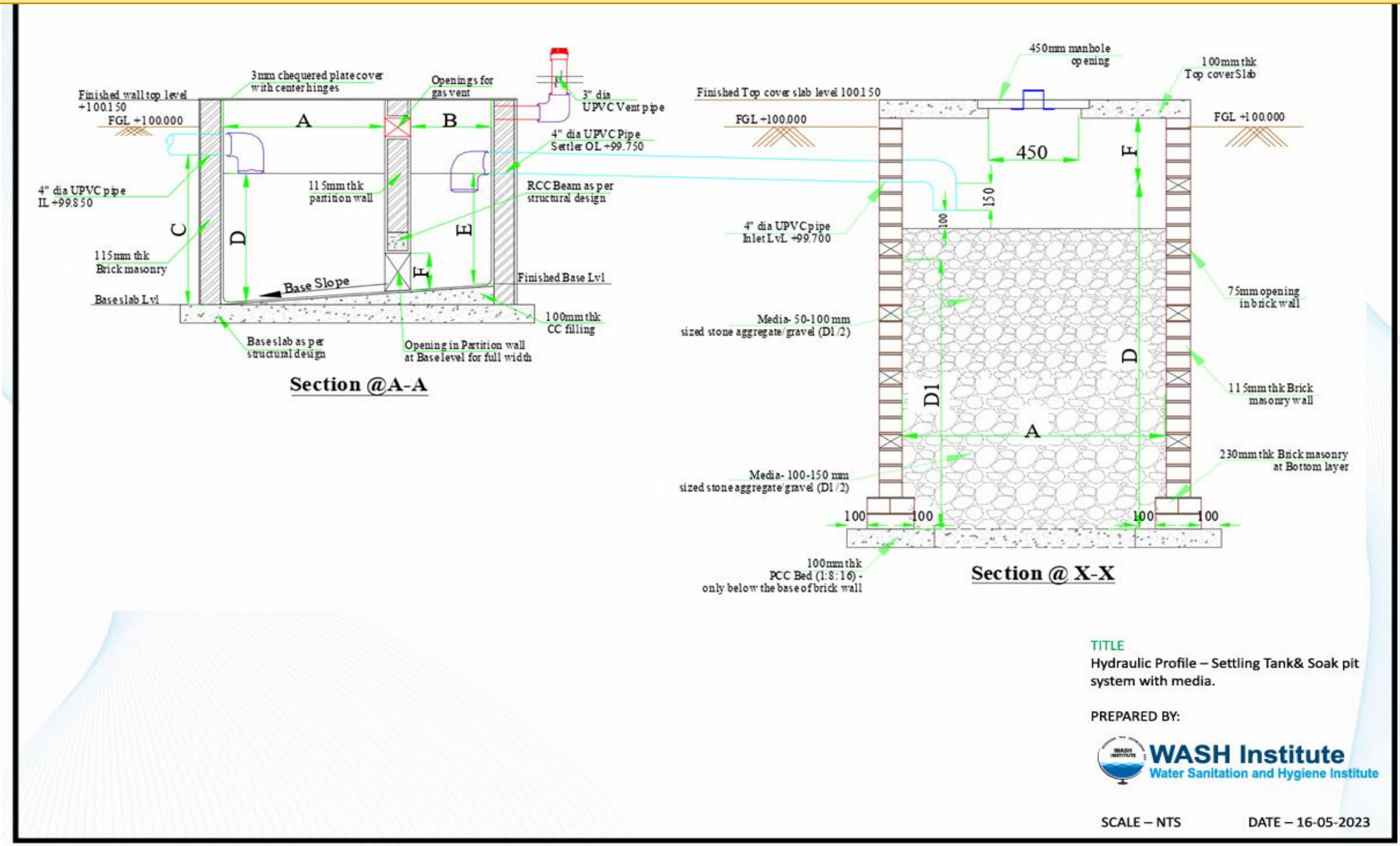
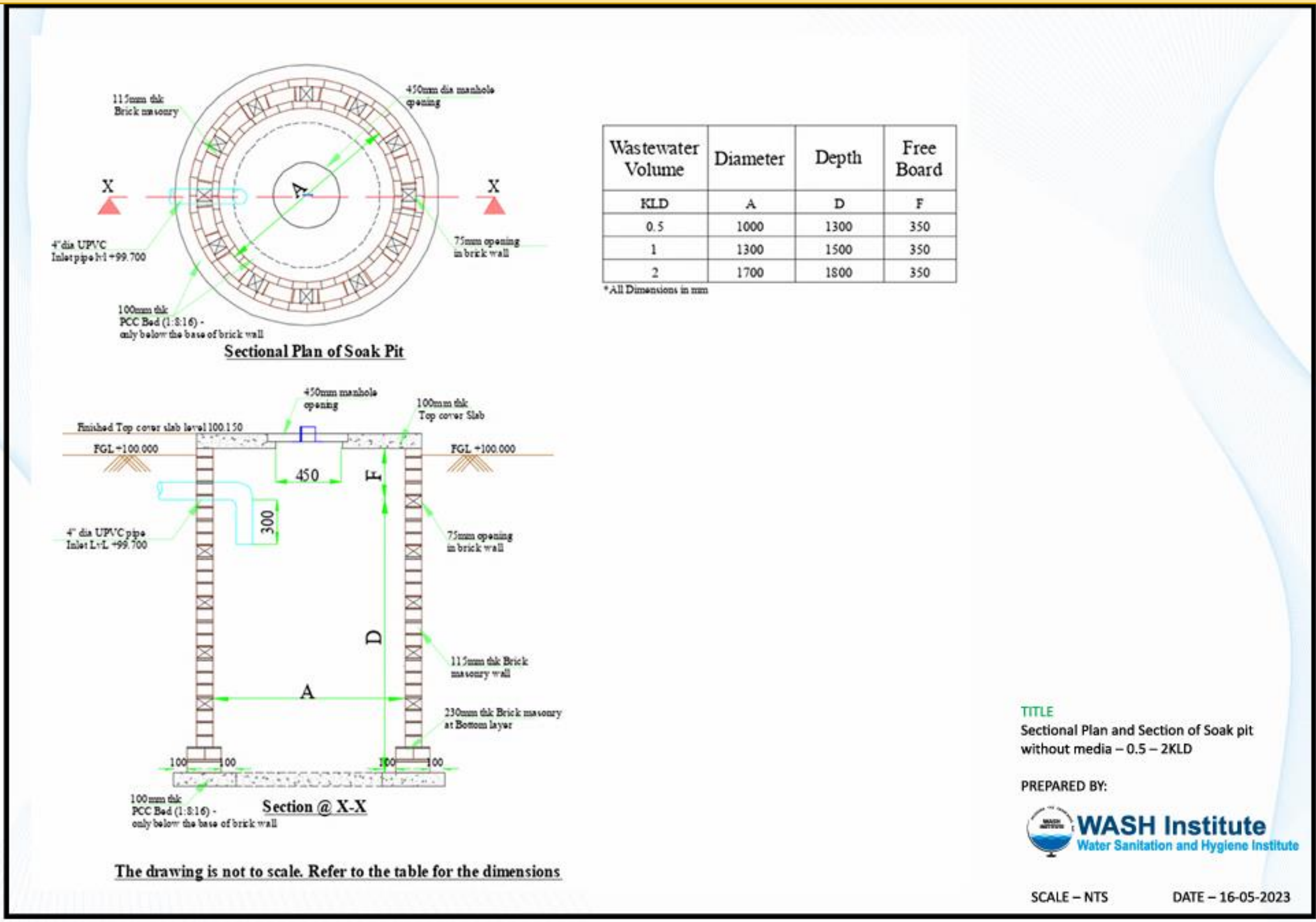


Figure 4.5 Hydraulic Profile of settling tank and Type B Soak Pit (with media)

SOAK PIT TYPE C (WITHOUT MEDIA) FOR HOUSEHOLD LEVEL DISPOSAL OF GREY WATER IN CLASS V AND CLASS VI TOWNS UNDER SBM U 2.0



Wastewater Volume	Diameter	Depth	Free Board
KLD	A	D	F
0.5	1000	1300	350
1	1300	1500	350
2	1700	1800	350

*All Dimensions in mm

Figure 4.6 Drawing of the Type 'C' Soak pit (without media) for 0.50 to 2.0 KLD flow

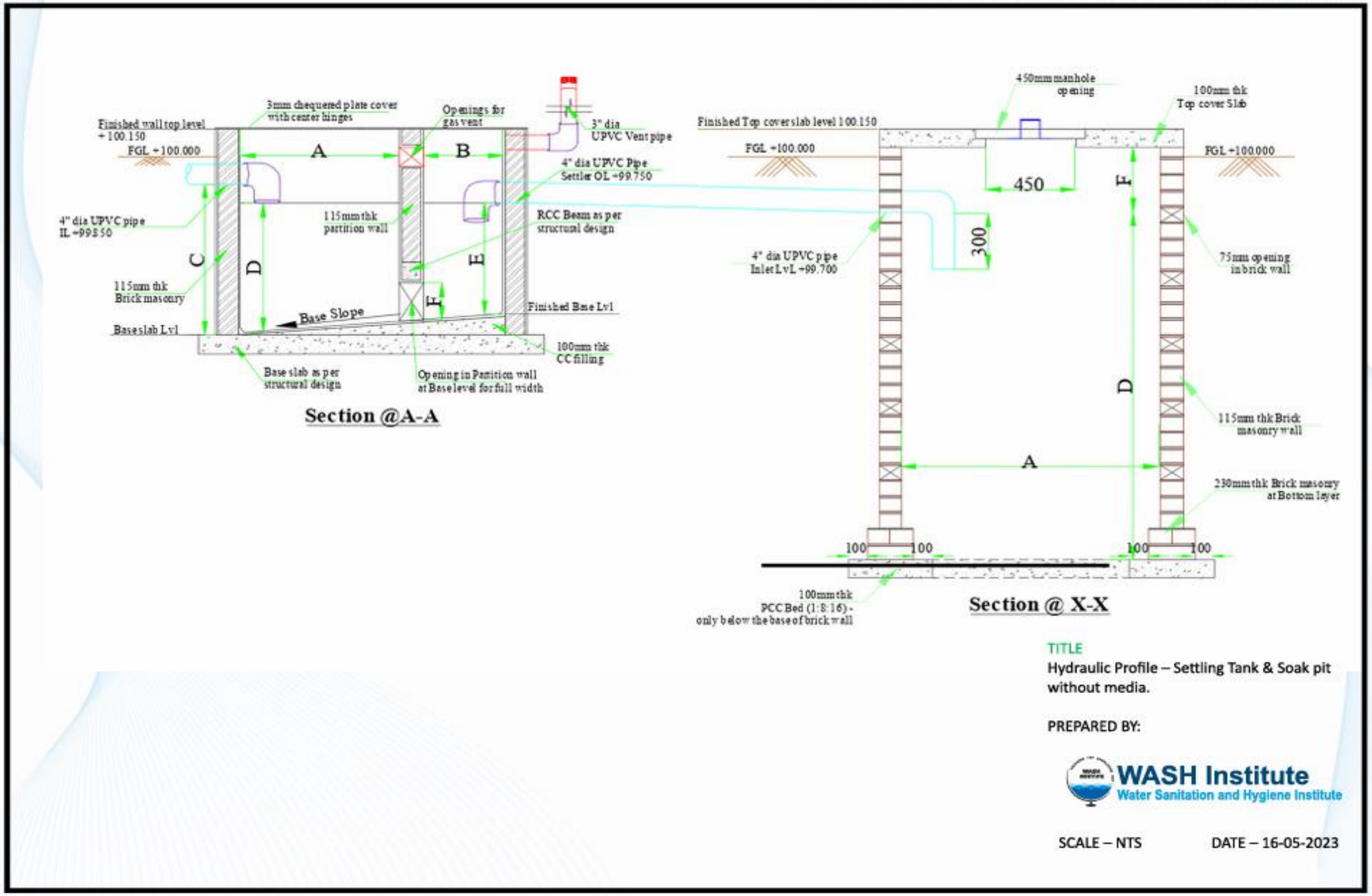
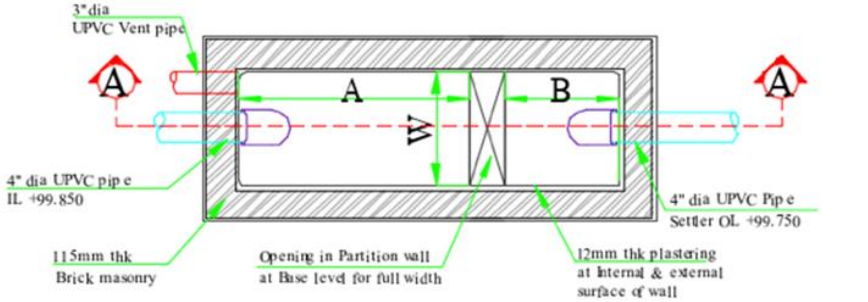
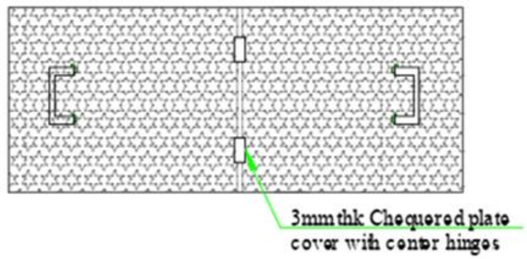


Figure 4.7 Hydraulic Profile of settling tank and Type C Soak Pit (without media)

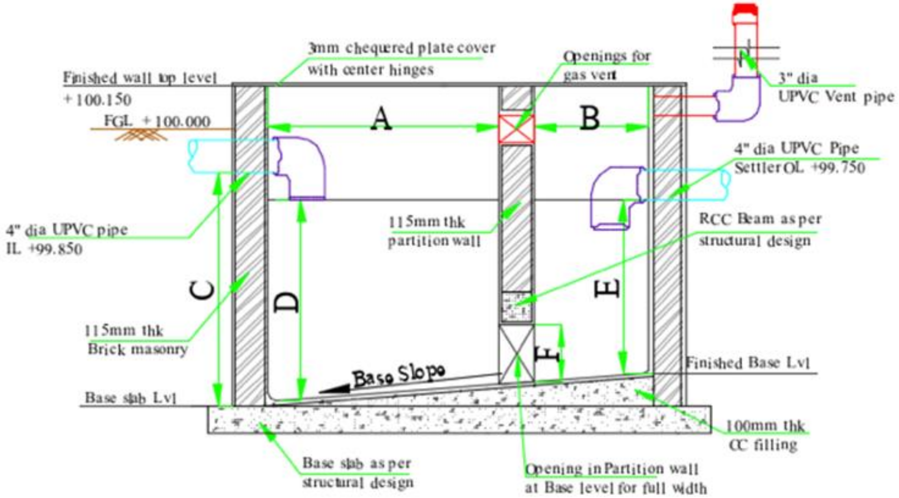
SETTLING TANK



Sectional Plan of Settler



Settler Cover



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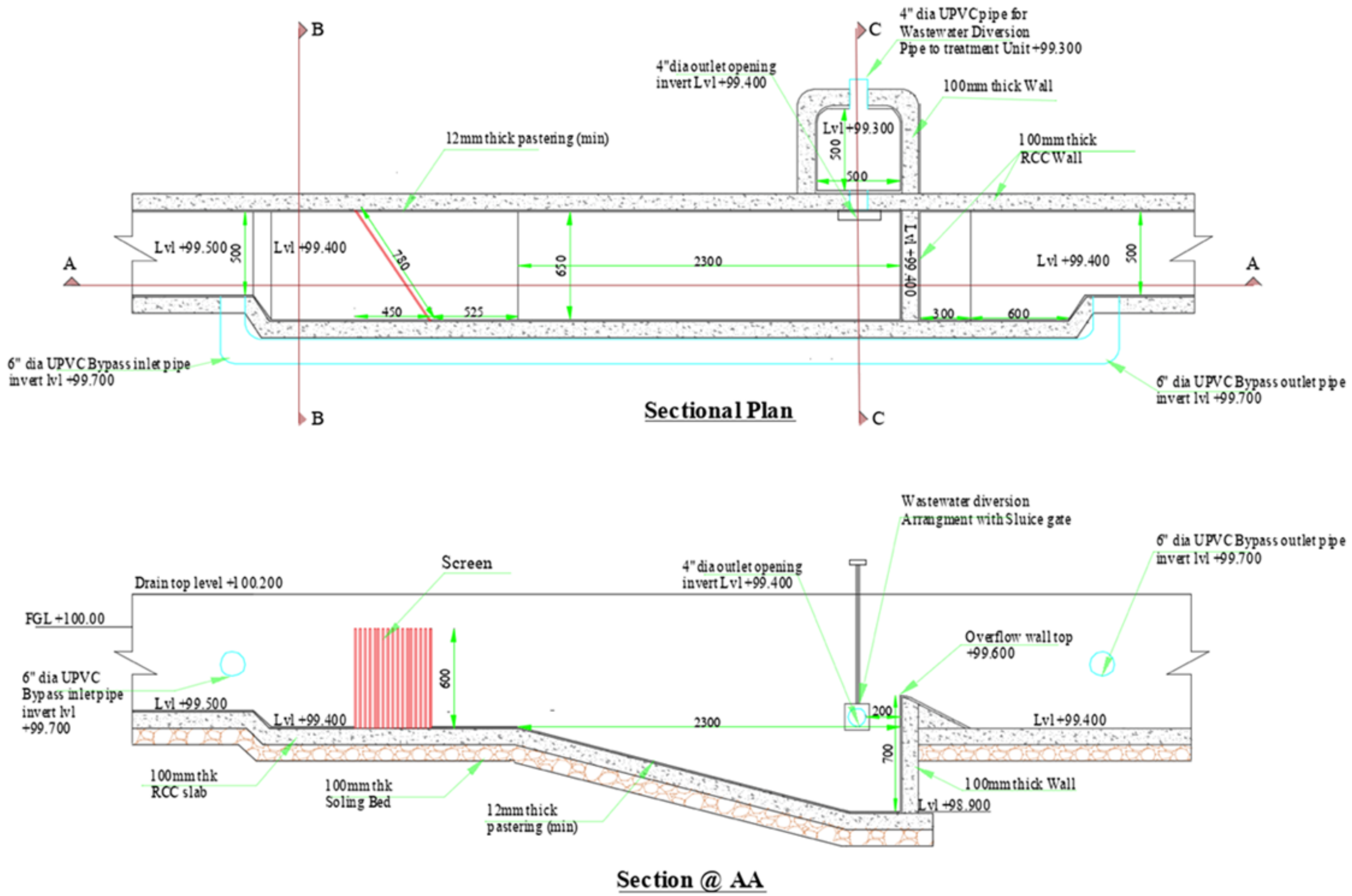
Section @A-A

Wastewater Volume	Length 1	Length 2	Width	Inlet side Depth	Hydraulic Depth	Outlet side Depth	Bottom opening
KLD	A	B	W	C	D	E	F
3	700	300	400	800	700	600	200
4	800	400	400	900	800	700	250
5	800	400	400	900	800	700	250
6	800	400	500	900	800	700	250
7	800	400	500	1000	900	800	300
8	900	500	500	1000	900	800	300
9	900	500	500	1100	1000	900	300
10	1000	500	500	1100	1000	900	300

*All Dimensions in mm

The drawing is not to scale. Refer the table for the domensions

Figure 4.8 Drawing of the settling tank for 3.0 to 10.0 KLD flow



Pre-Treatment Tank - 10KLD for GW Flow in the drain (including Screening system + Settling tank + I&D system)

Figure 4.9 Pre-Treatment tank for 10 KLD Community Level Type C (Without Media) Soak Pit

PRE-TREATMENT TANK SECTION FOR COMMUNITY LEVEL DISPOSAL OF GREY WATER IN CLASS V AND CLASS VI TOWNS UNDER SBM U 2.0

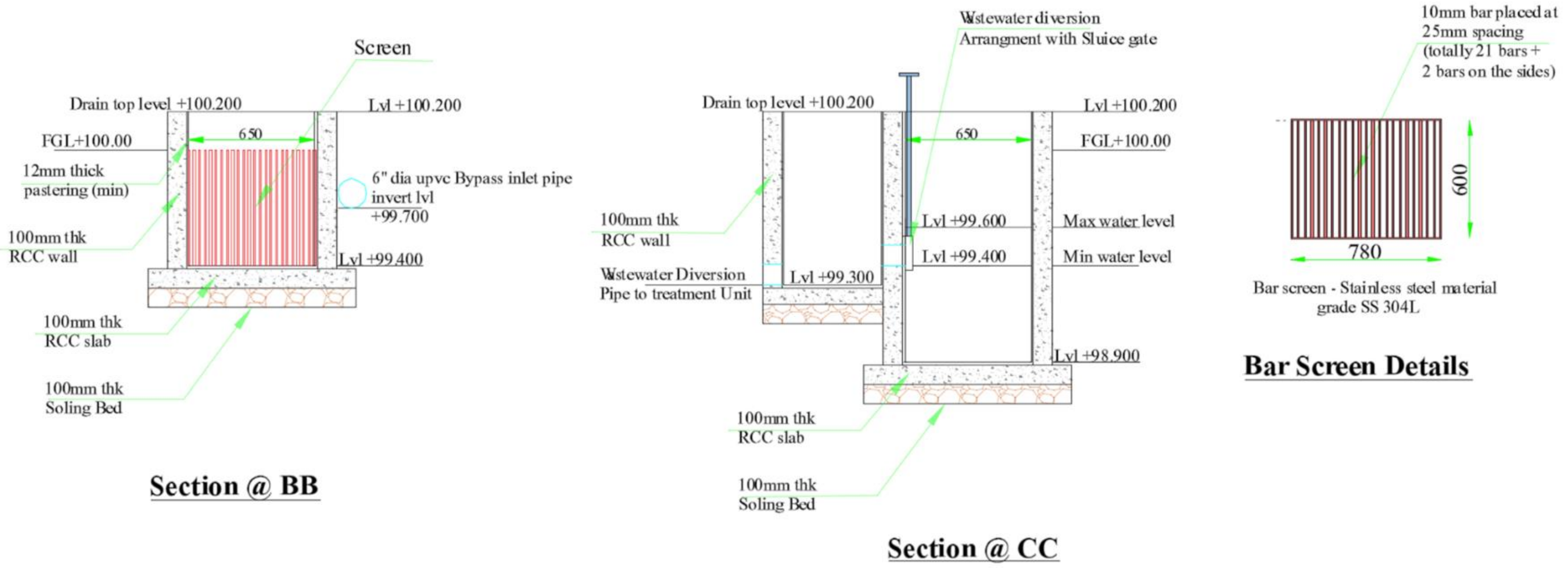
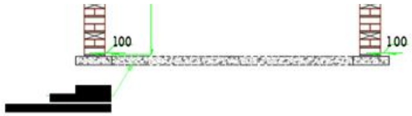


Figure 4.10 Sections BB and CC of Pre-Treatment tank for 10 KLD Community Level Type C (Without Media) Soak Pit

SOAK PIT / LEACH PIT SYSTEM WITHOUG FILTER MEDIA

Wastewater Volume	Diameter	Depth
KLD	A	D
3	1900	2100
4	2100	2300
5	2250	2500
6	2400	2700
7	2500	2800
8	2700	2900
9	2800	2900
10	3000	3000

* All Dimensions in mm



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The drawing is not to scale. Refer to the table for the dimensions

Figure 4.11 Drawing of the Type 'C' Soak pit (without media) for 3.0 to 10.0 KLD flow

HYDRAULIC PROFILE OF SETTLING TANK AND TYPE C (WITHOUT MEDIA) SOAK PIT FOR COMMUNITY LEVEL DISPOSAL OF GREY WATER IN CLASS V AND CLASS VI TOWNS UNDER SBM URBAN 2.0

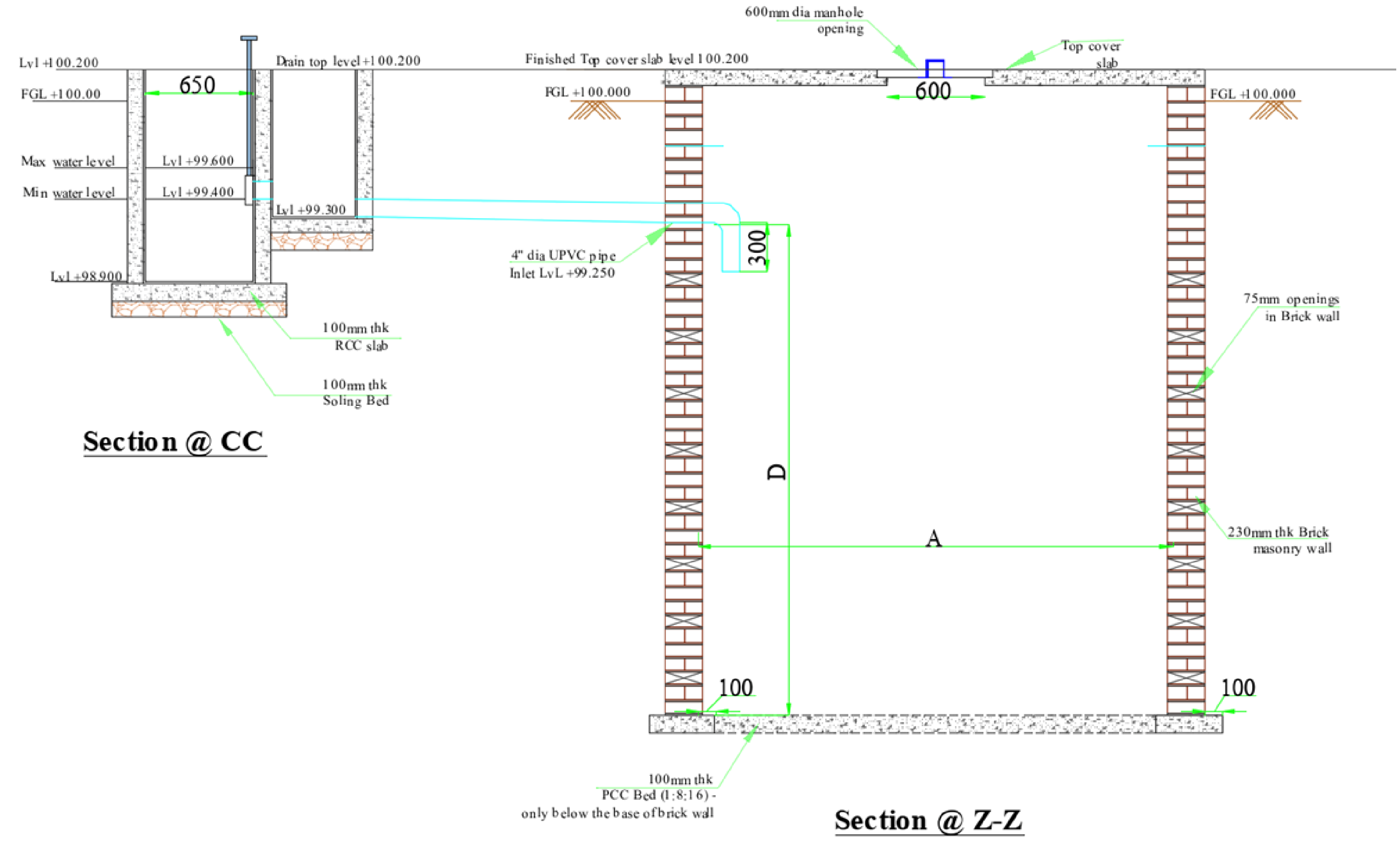

Figure 4.12 Hydraulic Profile of settling tank and Type C Soak Pit (without media) for 3.0 to 10.0 KLD flow

4.4 Design of CWL for On-Site Treatment and Disposal of Sullage from a small community

Having presented the type designs of nine technology options (combinations) from Section 2.4 to Section 2.12, an attempt is made in this part of the document to illustrate the nature based technology option consisting of employment of primary treatment (screens and grit chamber), secondary treatment (constructed wet land) followed by disposal of treated Used Water into a soak pit. The system is designed for a small community of 900 persons which generates a Used Water of 100 KLD.

For the following data:

No. of persons	=	900
Water supply	=	135 lpcd
Used Water generations as % of supply	=	80%
Groundwater infiltration	=	5%

Quantity of Used Water generated by the community = 900 x 135 x 0.8 x 1.05

= 102 060 litres / day

= 100 KLD (say)

4.4.1 Proposed On-Site Treatment and Disposal System

The proposed treatment and disposal system for 100 KLD Used Water consists of a wet well with pump into which the outfall of the drain is directed by using I & D device, coarse screen, medium screen, grit chamber and constructed wetland. The treated water is thereafter disposed by releasing it in a water body or water course or it is used in agriculture for irrigation purpose. Figure 4.13 shows the flowchart of the proposed system.

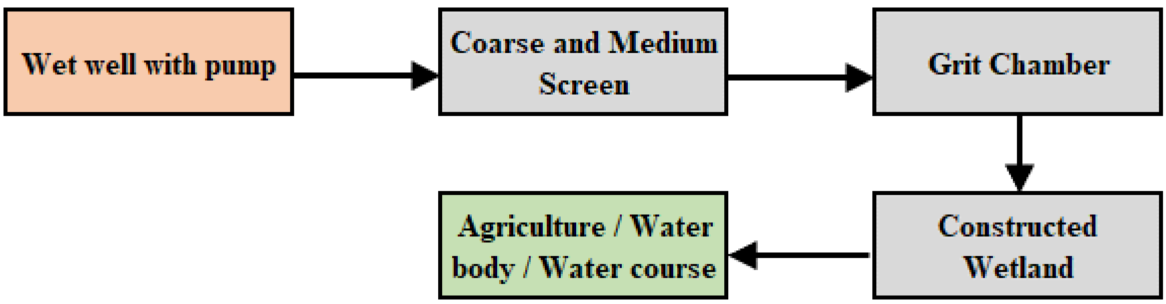

Figure 4.13 Flow chart of the proposed 1 KLD nature based treatment system

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4.4.1.1 Design of Wet Well with Pumping Machinery for 100 KLD STP

The wet well is the first component of any treatment plant and it is the chamber in which the sewer outfall empties. Usually it is circular, but rectangular wet wells are also employed. The depth of the wet well depends on the depth of the outfall sewer. Similarly, the lift above the ground level depends on the level at which the flow is discharged into the screen chamber. Submersible pump is employed for lifting the influent from the sump to the primary treatment chamber. The design of a 2 MLD wet well with pumping machinery has been presented in section 2.3. The same design is replicated below for a flow of 1 KLD.

The designs is based on practical assumptions for diameter of the outfall pipe, suction head, delivery head, minor losses, efficiency of the submersible pump set etc. In a real-time design, these inputs shall be replaced with the actual values based on the field conditions.

Table 4.7 Design calculations of 100 KLD wet well with Pumping Machinery

Sl. No.	Item	Description / value	Quantity	Units
1	Average flow	Capacity of the STP	0.1	MLD
2	Average flow	MLD x 1000	100.00	m^3/day
3	Average flow	(m^3/day) / (24 x 60 x 60)	0.00116	m^3/s
4	Peak factor	For a population of up to 20,000 as per CPHEEO Manual	3.00	
5	Peak flow	= Average flow x peak factor	0.0035	m^3/s
6	Peak flow	m^3/s x 60	0.21	m^3/ min
7	Volume of wet well	V = TQ/4 T = 15 minutes for small pumps (Reference: CPHEEO Manual)	0.78	m^3
8	Minimum depth	Below invert of sewer	2.00	m
9	Required Area of the well	Volume / depth	0.39	m^2
	Provide a circular wet well			

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10	Computed well diameter	d = (4 x Area / π)^0.5	0.70	m
11	Adopted well diameter		1.00	m
	Obtain Pump Capacity			
12	Diameter of invert pipe	Assumed	300.00	mm
13	Static head	H = suction head + Delivery head	10.00	m
14	Minor losses	10% of Manometric head	1.00	m
15	Total manometric head	Suction Head + Delivery Head + Minor losses	11.00	m
16	Efficiency	65%	0.65	
17	Pump Capacity	$P = \gamma Q H / \eta$	0.19	KW
18	Provided pump capacity	Provide 0.1 KW x 2 working 0.1 KW x 1 Standby	0.20	KW

Green cells: Input variable data to be provided Yellow cells: standard data from CPHEEO manual or other standards Red Cells: Design output

4.4.1.2 Design steps of Wet Well for 100 KLD STP

1. Average flow = 0.10 MLD

2. Average flow = MLD x 1000 = 0.1 x 1000 = 100 m^3 / day

3. Average flow = 100 / (24 x 60 x 60) = 0.00116 m^3/s

4. Peak factor = 3.00

   Peak factor: As per Table 3.2 of CPHEEO's Manual, the peak factor for contributory population up to 20, 000 is 3.0.

Table 3.2 from CPHEEO Manual

Table 3.2 Peak factor for Contributory Population

Contributory Population	Peak Factor
up to 20,000	3.00
Above 20,001 to 50,000	2.50
Above 50,001 to 7,50,000	2.25
above 7,50,001	2.00

Source: CPHEEO, 1993

5. Peak flow = average flow x peak factor = 0.00116 x 3 = 0.00347 m^3/s

6. Peak flow = 0.00347 x 60 = 0.2083 m^3/min

7. **Volume of wet well:** The volume of wet well is calculated based upon the formula V = T x Q / 4 furnished in section 4.6.6 of the CPHEEO manual.

   Where:

   V = Effective volume of wet well (in cubic meters)
   T = Time for one pump cycle (in minutes) = 15 minutes.
   Q = Pumping rate (cubic meters per minute)

   V = 15 x 0.20833 / 4 = 0.7811 m^3

8. **Minimum depth below invert of sewer:** The depth of the wet well required is governed by Table 4.1 in 4.6.6 of CPHEEO Manual. This is governed by the height of the submersible pump set and the floor clearance. In the present study the depth below the invert of the sewer is taken as 2 m.

9. Required area of the well = Volume / depth = 0.7811 / 2 = 0.3906 m^2

   Provide a circular wet well

10. Well diameter = $\sqrt{4 * 0.3906 / \pi}$ = 0.7054 m

11. Adopted well diameter (rounded off) = 1.00 m
    The minimum diameter of the well is adopted as 1.00 in order to accommodate maximum of three submersible pumps, including standby pump.

12. Assume diameter of the invert pipe = 300 mm

13. **Static Head:** The static head is defined as the sum of suction head and delivery head. For a centrifugal pump it represents the vertical distance from the water level in the sump to the maximum height water is lifted. In case of a submersible pump it is taken

as the level from the centre of the submersible pump to the maximum height up to which water is lifted. The concept of static head in terms of suction and delivery heads is presented in Figure 2.2.

Static Head = suction head + delivery head = 10 m

14. **Minor Losses:** The minor head losses are taken as 10% of the static head. If an exact estimate of head loss is available, the same can be used.

15. **Total Manometric Head:** The manometric head is the sum of static head and losses.
Static Head = Suction head + delivery head = vertical distance from the centre of the submersible pump to the maximum lift = 10 m
Head losses = 10% of static head = 0.1 x 10 = 1 m
Manometric Head = 10 + 1 = 11 m.

16. **Overall Efficiency:** From Table 4.3 of CPHEEO Manual, the overall efficiency for submersible pump is 65%. The efficiency is maximum for horizontally mounted centrifugal pump. Appropriate value may be taken from the table.

Table 4.3 from CPHEEO Manual

Table 4.3 Efficiencies of pumps to be adopted for design purposes

No.	Type of Pump Set	Efficiency
1	Horizontal foot mounted centrifugal pump sets	0.85
2	Vertical shaft centrifugal pump sets	0.8
3	Submersible pump sets	0.65
4	Positive displacement pump sets	0.40

Note: *It is suggested that appropriate value of efficiency may be taken as suggested in the pump manufacturer manual.*

17. **Pump capacity:** The capacity of the hydraulic pump is obtained for the dry weather flow as per 4.5.4 of CPHEEO manual. The general practice is to provide 3 pumps for a small capacity pumping station comprising of (a) 1 pump of 1 DWF, (b) 1 of 2 DWF and (c) 1 of 3 DWF capacity. Alternatively, the number of pumps can also be chosen to be in multiples of DWF flow and provide a 100% standby capacity for peak flow.

Specific weight of sullage: Specific gravity of sewage is not mentioned in the CPHEEO manual. In literature it is mentioned as slightly more than 1, say 1.2. In the present calculation the specific gravity of sewage is taken as 1.

Hence Specific Weight = γ = 9810 N / m^3

P = γ Q H_m / η Watts

P = 9810 x 0.00116 x 11 / 0.65 = 192.5778 W = 0.1926 KW

18. Provided pump capacity, P = say 0.2 KW.

Provide 3 Nos. 0.1 KW submersible pumps, 2 Working + 1 Standby thereby ensuring 50% standby capacity for peak flow as per the clause of CPHEEO manual.

4.4.1.3 Design of Coarse Screen for 100 KLD STP

A coarse screen plays a critical role in the initial stages of the treatment process by removing large objects and debris from the influent sewage. This includes items like plastics, wooden sticks, rags, and other large solids that could potentially damage downstream equipment or interfere with the treatment process.

The most popular type of coarse screens are the bar screens which consist of vertical or inclined bars sapced 25 mm apart to allow wastewater to flow through while capturing large objects.

The design of coarse screen for 100 KLD STP, based on CPHEEO's manual is presented in Table 4.8. Design steps are similar to the calculations presented in section 2.3.2.1.

Table 4.8 Design of coarse screen for 100 KLD STP

S. No.	STP Capacity	(MLD)	0.1	
1	Average Flow		0.10000	MLD
2	Average Flow		0.00116	m^3/s
3	Peak Factor		3.00000	
4	Peak Flow		0.00347	m^3/s
5	Coarse Screen Opening		25.00000	mm
6	Depth of water in screen		0.30000	m

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7	velocity through screen	0.75000	m/sec
8	Area of screen	0.00463	m^2
9	Angle of inclination with the horizontal	45.00000	degree
10	SIN (Angle of Inclination)	0.70711	
11	Free Board	0.60000	m
12	Length of screen	1.27279	m
13	Provided length of the screen	1.30000	m
14	Total width of the screen opening	0.01543	m
15	Provided width of screen opening	0.30000	m
16	No of opening	9.00000	nos
17	Number of bars	8.00000	nos
18	Let width of each bar be (10 x 50 mm flats)	10.00000	mm
19	Total width of screen	305.00000	mm
20	Provided width of the screen	300.00000	mm
21	Let width of each side wall be	50.00000	mm
22	Total width of channel: Provide	400.00000	mm
23	Size of Coarse Screen Channel - Length	3900.71698	mm
24	Provided coarse screen channel length	4.00000	m
25	Appraoch velocity in channel u/s of Screen	0.02894	m/s
26	Velocity through Screen	0.05144	m/s
27	Head Loss No Clogging	0.00013	m
28	Velocity when 50% clogging	0.10288	m/s
29	Head Loss when 50% clogging	0.00071	m

Green cells: Input variable data to be provided Yellow cells: standard data from CPHEEO manual or other standards Red Cells: Design output

4.4.1.4 Design of Medium Screen for 100 KLD STP

In sewage treatment plants, a medium screen, also known as intermediate screen, complements the functions of coarse screen by capturing smaller particles and materials that could still potentially interfere with downstream treatment processes if not removed. Medium screens typically have smaller gaps or openings compared to coarse screens, usually in the range of 6-10 mm. This allows them to effectively capture smaller debris such as smaller plastics, fibers, and other suspended solids.

The design of medium screen for 100 KLD STP, based on CPHEEO's manual is presented in Table 4.9. Design steps are similar to the calculations presented in section 2.3.2.2.

Table 4.9 Design of Medium screen for 100 KLD STP

S. No.	STP Capacity	100	KLD
1	Average Flow	0.10000	MLD
2	Average Flow	0.00116	m^3/s
3	Peak Factor	3.00000	
4	Peak Flow	0.00347	m^3/s
5	Velocity through screen (Assumed)	0.80000	m/s
6	Area required	0.00434	m^2
7	Depth of flow (Assumed)	0.30000	m
8	Width of opening	0.01447	m
9	Clear opening between adjacent bars of screen	12.00000	mm
10	Bars thickness of screen	10.00000	mm
11	No of Openings	2.00000	Nos.
12	No of Bars	1.00000	Nos.
13	Width of the screen	34.00000	mm
14	Angle of inclination with the horizontal	75.00000	degree
15	Taking width of screen	300.00000	mm
16	No. of opening for adopted width of screen	15.00000	Nos.
17	No. of bars (10 mm thickness)	14.00000	Nos.
18	Free Board	0.60000	m
19	Sin (Angle of Inclination with horizontal)	0.96596	
20	Inclined length of Screen	0.93172	m
21	Length of medium screen chamber	3241.15132	m

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22	Chamber length adopted for medium screen	3.50000	m
23	Width of each side wall	50.00000	mm
24	Total width of channel	0.40000	m
25	Velocity in Channal u/s of Screen	0.03858	m/s
26	Velocity through Screen	0.06430	m/s
27	Head Loss through screen	0.00019	m
28	Velocity through screen when 50% clogged	0.12860	m/s
29	Head Loss when 50% clogging	0.00110	m/s

Green cells: Input variable data to be provided Yellow cells: standard data from CPHEEO manual or other standards Red Cells: Design output

4.4.1.5 Design of Grit Chamber for 100 KLD STP

Grit chamber is employed after the medium screen to remove grit (sand, gravel, and other heavy solid particles) from wastewater. In the present study horizontal flow grit chamber is adopted. The detention time is usually 1 to 5 minutes allowing 100% of the grit particles of 15 mm size to settle. The design output is presented in Table 4.10 and the design calculations are similar to the calculatons presented in section 2.3.2.3.

The summary of the design output of coarse screen, medium screen and grit chamber is presented in Table 4.11.

Table 4.10 Design of grit chamber for 100 KLD STP

S. No.	**STP Capacity**	**0.1**	**MLD**
1	Average Flow	0.10	MLD
2	Average Flow	0.00116	m^3/s
3	Peak Factor	3.00	
4	Peak Flow :	0.00347	m^3/s
5	Basic formula for settlement of grit	$Q/A=V_S$ x $n/((1-\eta)^{-n}-1)$	
	Where-η-Desired efficiency of removal of grit particle		
	V_S -settling velocity of minimum size of grit particle to be removed		

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	Q/A--Design surface overflow rate applicable for grit chamber to be designed		
	n-an index which a measure of the basin performance		
6	Here η value taken	75.00	%
7	Say	0.75	
8	Here V_S value taken	1528.97	$m^3/m^2/day$
9	Here n value taken-1/8(for very good performance)	0.125	
10	Hence surface over flow rate(Q/A)=(Vs*n)/(((1-η)^-n)-1)	1010.11	$m^3/m^2/day$
11	Say	1010.00	$m^3/m^2/day$
12	Peak flow($m^{3/}$day)	300.00	(m^3/day)
13	No. of grit chambers proposed	1.00	Nos.
14	Peak flow per chamber	300.00	(m^3/day)
15	Hence area required for peak flow	0.297	m^2
16	Detention time	1.00	minute
17	Choose width of the tank	0.75	m
18	Choose depth of flow	0.30	m
19	Particle size for critica displacement velocity	0.15	mm
20	Critical displacement velocity	0.1971	m/s
21	Horizontal velocity in the chamber	0.0154	m/s
22	Free board	0.30	m
23	Provison of space for grit	0.30	m
24	Length of the tank	0.93	m
25	Adopted legnth of the channel	1.00	m
26	Total depth of the tank	0.90	
27	This gives detention time	64.80	Seconds

Green cells: Input variable data to be provided Yellow cells: standard data from CPHEEO manual or other standards Red Cells: Design output

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Table 4.11 Summary of the design of primary treatment units for 100 KLD STP

S. No.	Component	Particulars	Size
1	Coarse Screen Chamber	Length	4.0 m
2		Width	0.40 m
3		Depth	0.9 m
4		Screen size	1.3 m x 0.3 m
1	Medium Screen Chamber	Length	3.5 m
2		Width	0.4 m
3		Depth	0.9 m
4		Screen size	1.0 m x 0.3 m
1	Grit Chamber	Length	1.00 m
2		Width	0.75 m
3		Depth	0.9 m

4.4.1.6 Design of Constructed Wetlands for 100 KLD STP

Constructed wetlands are engineered systems which use natural processes involving wetland vegetation, soils and their associated microbial action to treat the Used Water. Detailed design steps of constructed wetlands are persetned in section 2.5.3.

The present design is for grey water / sullage from the household sinks, baths, washing areas and septic tank outlets etc., except toilets. In various literatures, the B.O.D. of sullage is mentioned as ranging from 100 to 300 mg/l (Manual: Greywater Management, Department of Drinking Water and Sanitation, Ministry of Jal Shakti, GoI, July 2021). In the present study, the post primary treatment BOD of the sullage is taken as 100 mg/l. It is understood that the designer of the facility will use the sullage BOD as obtained in the BOD test undertakne in a laboratory.

The poposed system using nature-based technology is intended for on-site disposal by construction of a soakpit or by using the treated water from the constructed wetland for agriculture. The design of the 100 KLD constructed wetland is presented in Table 4.12.

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Table 4.12 Design of constructed wetland for 100 KLD STP

DESIGN OF 100 KLD HORIZONTAL GRAVEL FILTER CONSTRUCTED WETLAND				
S. No.	**Design parameter**	**Notation / Formula used**	**Assumed / computed value**	**Units**
1	Design flow of Sewage	$Q_{average}$	0.1	MLD
2	Average flow m^3/day	$Q_{average}$ (MLD) x $10^6/10^3$	100	m^3/day
3	BOD of the influent waste water	Corresponds to the BOD of the effluent from the ABR	100	mg/l
4	BOD of the effluent from the constucted wet land	CPHEEO norms	30	mg/l
5	Temperature	Minimum temeprature of the year	14.5	^oC
6	Height of the filter	Adopted standard value	0.6	m
7	Slope of the wetland, S	Adopted standard value	1	%
8	Voids in the gravel (Porosity)	Vide Table 3.4 of US EPA manual	39	%
9	Ks, Hydraulic conductivity of the medium (Coarse Sand)	Vide Table 3.4 of US EPA manual	480	m^3/m^2-day
10	Size of gravel / grains	Vide Table 3.4 of US EPA manual maximum 10% grain size	2	mm
11	K_{20} = Rate constant at 20^oC	Vide Table 3.4 of US EPA manual	1.35	DL
12	Check the product (K_s S)	Should be < 8.6	4.8	OK
13	K_T = Rate constant at design temperature	$K_T = K_{20}\ (1.1)^{T-20}$	0.80	DL
14	Required cross sectional area of the bed	$A_c = Q / (K_s\ S)$	20.83	m^2

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15	Determine the bed width, W	$W = A_c / d$	34.72	m
16	Adopted bed width, W		34.00	m
17	Required surface area	$A_s = (Q (\ln C_o - \ln C_e)) / (K_T\ d\ n)$	643.76	m^2
18	Determine the bed length L	$L = A_s / W$	18.54	m
19	Adopted bed length, L		19.00	m
20	Hydraulic Retention time	$V_v / Q = L\ W\ d\ n / Q$	1.51	days

The following is the size of the constructed wetland to treat 100 KLD of sullage with influet BOD_5 of 100 mg/l:

Length = 19 m

Width = 34 m

Depth = 0.6 m

The BOD of treated water at the outlet of the constructed wet land is 30 mg/l as per norms.

4.4.1.7 Design of Chlorination Tank for 100 KLD STP

1. The design process of chlorination tank is very simple. The size of the tank is obtained by considering the detention period as 30 minutes.

2. Flow = 100 KLD

3. = 100 m^3/day

4. Detention period = 30 minutes

5. Volume of tank = $100 \times 10^3 \times 0.5 / (10^3 \times 24)$

   = 2.0833 m^3

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6. Let depth be = 1.0 m

7. Free board = 0.3 m

8. Total depth of tank = 1.3 m

9. Area of tank = Volume / depth

= 2.0833 / 1.3

= 1.60 m^2

10. Required side of the tank = $\sqrt{1.60}$

= 1.26 m

11. Hence provide a tank of size 1.3 x 1.3 x 1.3 m

4.5 Cost Estimates

The cost estimates of proposed treatment systems for safe disposal of grey water (septage) consisting of Used Water from bathroom / kitchen / wash areas and the effluent from on-site sanitation systems consisting of septic tanks / Imhoff tanks is presented below.

The designs belong to guidelines prepared by WASH Institute for Liquid Waste Management under Swachh Bharat Mission Grameen – Punjab. It is felt that these designs can be adopted in the urban areas of Class V and Class VI towns with population of less than 10, 000. Cost estimates are not included in the publication prepared by WASH. Hence an attempt is made herein to present the cost estimates for the guidance of the ULBs of Class V and Class VI towns.

4.5.1 Cost Estimate of Settling Tank with Type – B Soak Pit (with media) for household level Management / Disposal

These cost estimates (CAPEX) refer to the design data of settling tank presented in Table 4.1, Type B soak pit (with media) presented in Table 4.2 and the subsequent drawings presented in Figures 4.3 to 4.5. The outcome pertains to three capacities, namely, 0.25-0.5 KLD, 1 KLD and 2 KLD and is presented in Table 4.7.

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4.5.2 Cost Estimate of Settling Tank with Type–C Soak Pit (without media) for Household Level Management / Disposal

These cost estimates (CAPEX) refer to the design data of settling tank presented in Table 4.1, Type C soak pit (without media) presented in Table 4.3 and the subsequent drawings presented in Figure 4.3, Figure 4.6 and Figure 4.7. The outcome pertains to three capacities, namely, 0.5 KLD, 1 KLD and 2 KLD and is presented in Table 4.8.

4.5.3 Cost Estimate of Settling Tank and Pre-Treatment tank with Type – C soak pit (without media) for Community level disposal

These cost estimates (CAPEX) refer to the design data of settling tank presented in Table 4.4, and Type C soak pit (without media) presented in Table 4.6, followed by the subsequent drawings presented in Figure 4.8, Figure 4.11 and Figure 4.12. The design range pertains to eight capacities; 3 KLD to 10 KLD at intervals of 1 KLD and the outcome of settling tank and Type C soak pit is presented in Table 4.9.

Sample design drawing of 10 KLD pre-treatment tank presented in Figure 4.9 and Figure 4.10 and the corresponding cost estimate is presented in Table 4.10.

4.5.4 Cost Estimate of Constructed Wetland for on-site treatment and disposal of 100 KLD sullage of a small community

The CaPex of the system for treating 100 KLD of sullage by employing constructed wetland as the secondary secondary treatment is preseted in Table 4.11. The components consist of wet well with pump, coarse screen, medium screen, grit chamber and constructed wetland.

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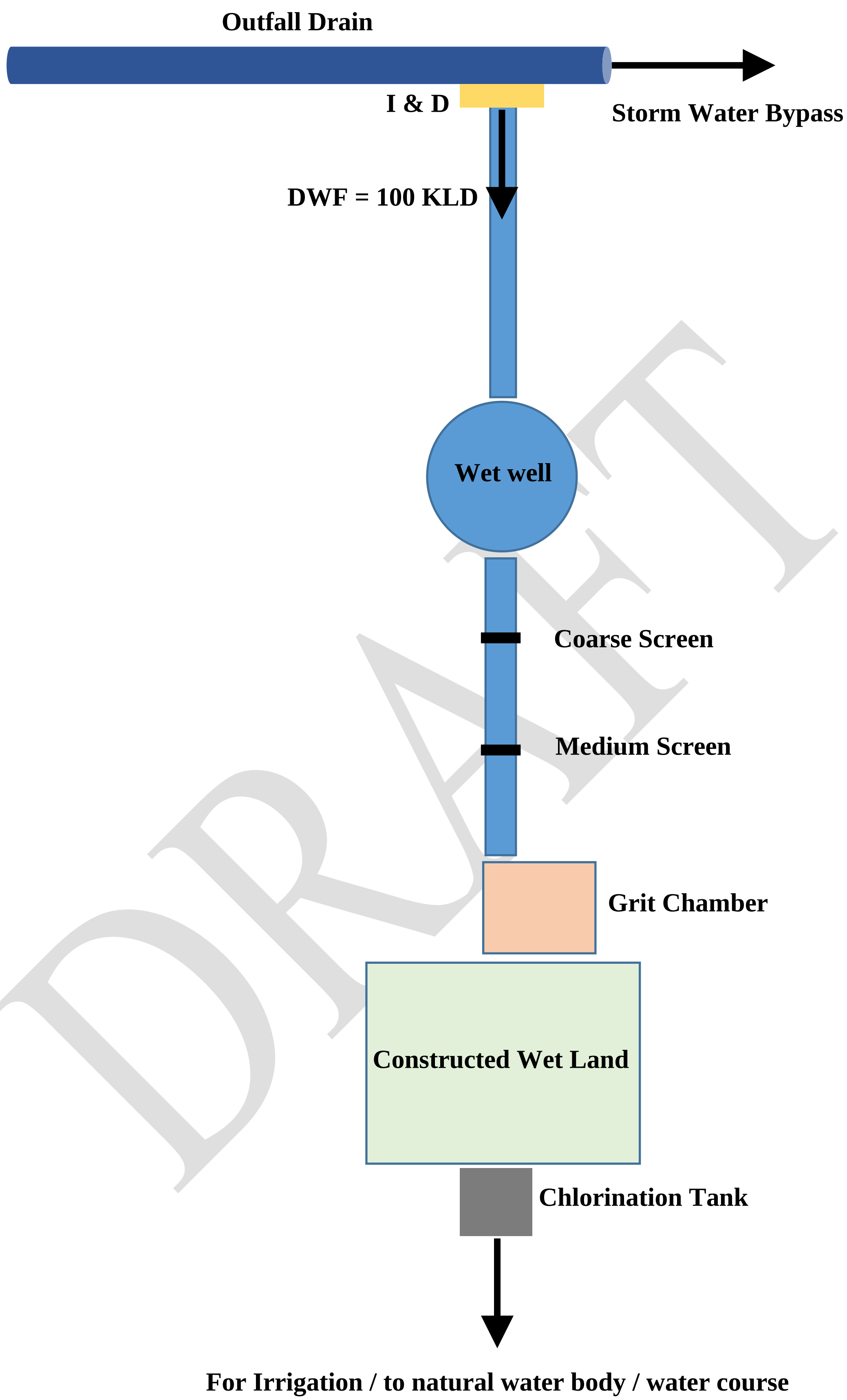

Figure 4.14 Arrangement of Components of the 100 KLD Constructed Wetland STP

Table 4. 13 CAPEX of settling tanks (0.25-0.50, 1 and 2 KLD) and Type – B Soak pits (with media)

Household level disposal through settling tank and Soak Pit Type - B (With Media)																
S. No.	SETTLING TANK									SOAK PIT						Total cost (Rs.)
	Waste water Volume	Length 1	Length 2	Inlet side Depth	Hydraulic Depth	Outlet side Depth	Bottom Opening	Width	Cost in Rs.	Waste Water Volume	Diameter	Depth	Media Depth	Free Board	Cost in Rs.	
	KLD	**A**	**B**	**C**	**D**	**E**	**F**	**W**		**KLD**	**A**	**D**	**D1**	**F**		
1	0.25-0.5	0.40	0.30	0.50	0.40	0.30	0.10	0.3	**7, 939**	0.25	1.00	1.30	1.15	0.35	**12, 124**	**20,063**
2	1	0.60	0.30	0.60	0.50	0.40	0.20	0.3	**9, 004**	0.50	1.30	1.60	1.45	0.35	**18, 087**	**27,091**
3	2	0.70	0.30	0.60	0.50	0.40	0.20	0.3	**9, 509**	1.00	1.60	2.10	1.95	0.35	**27, 505**	**37,000**

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Table 4. 14 CAPEX of settling tanks (0.50, 1 and 2 KLD) and Type – C Soak pits (without media)

Household level disposal through settling tank and Soak Pit Type - C (Without Media)																
S. No.	SETTLING TANK									SOAK PIT						Total cost (Rs.)
	Waste water Volume	Length 1	Length 2	Inlet side Depth	Hydraulic Depth	Outlet side Depth	Bottom Opening	Width	Cost in Rs.	Waste Water Volume	Diameter	Depth	Media Depth	Free Board	Cost in Rs.	
	KLD	A	B	C	D	E	F	W		KLD	A	D	D1	F		
1	0.5	0.4	0.3	0.5	0.4	0.3	0.1	0.3	**7, 939**	0.50	1.00	1.30	0.00	0.35	**11, 039**	**18,978**
2	1	0.6	0.3	0.6	0.5	0.4	0.2	0.3	**9, 004**	1	1.30	1.50	0.00	0.35	**15, 349**	**24,353**
3	2	0.7	0.3	0.6	0.5	0.4	0.2	0.3	**9, 509**	2	1.70	1.80	0.00	0.35	**22, 802**	**32,311**

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Table 4. 15 CAPEX of settling tanks (3.0 to 10.0 KLD) and Type – C Soak pits (without media)

Community level disposal through settling tank, Pre-Treatment tank and Soak Pit Type - C (Without Media)																
	SETTLING TANK									SOAK PIT						
S. No.	Waste water Volume	Length 1	Length 2	Inlet side Depth	Hydraulic Depth	Outlet side Depth	Bottom Opening	Width	Cost in Rs.	Waste Water Volume	Diameter	Depth	Media Depth	Free Board	Cost in Rs.	Total Cost (in Rs.)
	KLD	A	B	C	D	E	F	W		KLD	A	D	D1	F		
1	3	0.7	0.3	0.8	0.7	0.6	0.2	0.4	**12,185**	3	1.90	2.10	0.00	0.75	**52, 050**	**64,235**
2	4	0.8	0.4	0.9	0.8	0.7	0.25	0.4	**13,492**	4	2.10	2.30	0.00	0.75	**35, 593**	**49,085**
3	5	0.8	0.4	0.9	0.8	0.7	0.25	0.4	**13,492**	5	2.25	2.50	0.00	0.75	**40, 612**	**54,104**
4	6	0.8	0.4	0.9	0.8	0.7	0.25	0.5	**14, 875**	6	2.40	2.70	0.00	0.75	**46, 098**	**60,973**
5	7	0.8	0.4	1	0.9	0.8	0.3	0.5	**14, 933**	7	2.50	2.80	0.00	0.75	**49, 837**	**64,770**
6	8	0.9	0.5	1	0.9	0.8	0.3	0.5	**16, 355**	8	2.70	2.90	0.00	0.75	**56, 018**	**72,373**
7	9	0.9	0.5	1.1	1	0.9	0.3	0.5	**16, 419**	9	2.80	2.90	0.00	0.75	**58, 774**	**75,193**
8	10	1	0.5	1.1	1	0.9	0.3	0.5	**17, 205**	10	3.00	3.00	0.00	0.75	**69, 188**	**86,393**

Table 4. 16 CAPEX of 10 KLD Pre-Treatment tank

Community level disposal through settling tank, Pre-Treatment tank and Soak Pit Type - C (Without Media)			
S. No.	PRE-TREATMENT TANK		
	Waste water Volume KLD	Reference Design Drawing	Cost in Rs.
1	10	As per Table 4.5 and drawings in Figure 4.9 and Figure 4.10.	**1, 63, 038.86**

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Table 4. 17 CaPex of 100 KLD Constructed Wetland for onsite treatment of sullage

CAPACITY - 100 KLD			
TECHNOLOGY – CONSTRUCTED WETLAND			
S.No.	**Components**	**Sizes**	**Cost in Lakhs Rs.**
1	CC Drain from the outfall drain to the collection well / wet well	0.3 m x 0.9 m section 5 m long	0.25
2	Collection well	1 m diameter and 3.5 m depth	4.41
3	Pump sets for the collection well	2 Nos. 4 KW Pumps	1.12
4	Coarse screen chamber	0.3 m x 0.9 m section 4 m long	0.30
5	Medium screen chamber	0.30 m x 0.90 m section 3.5 m Length	0.25
6	Coarse Screen	1.3 m x 0.3 m	0.11
7	Medium Screen	1.0 m x 0.3 m	0.15
8	I & D structure / Gate	1 No	0.10
9	Grit Chamber	1.0 m x 0.75 m x 0.9 m	0.19
10	CC Drain from Grit Chamber to the Constructed wet lands	0.30 m x 0.90 m section 5 m Length	0.26
11	Constructed Wetland	(1No.) 34.0 m x 19.0 m x 0.6 m	6.78
12	Chlorination Tank	1.3 m x 1.3 m x 1.3 m	4.26
13	Cost for 100 KLD in Lakhs of Rs.		18.18 Lakhs
14	**Cost Per 100 KLD in Cores of Rs.**		**0.18 Crores**

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