Design of Wastewater Treatment plant

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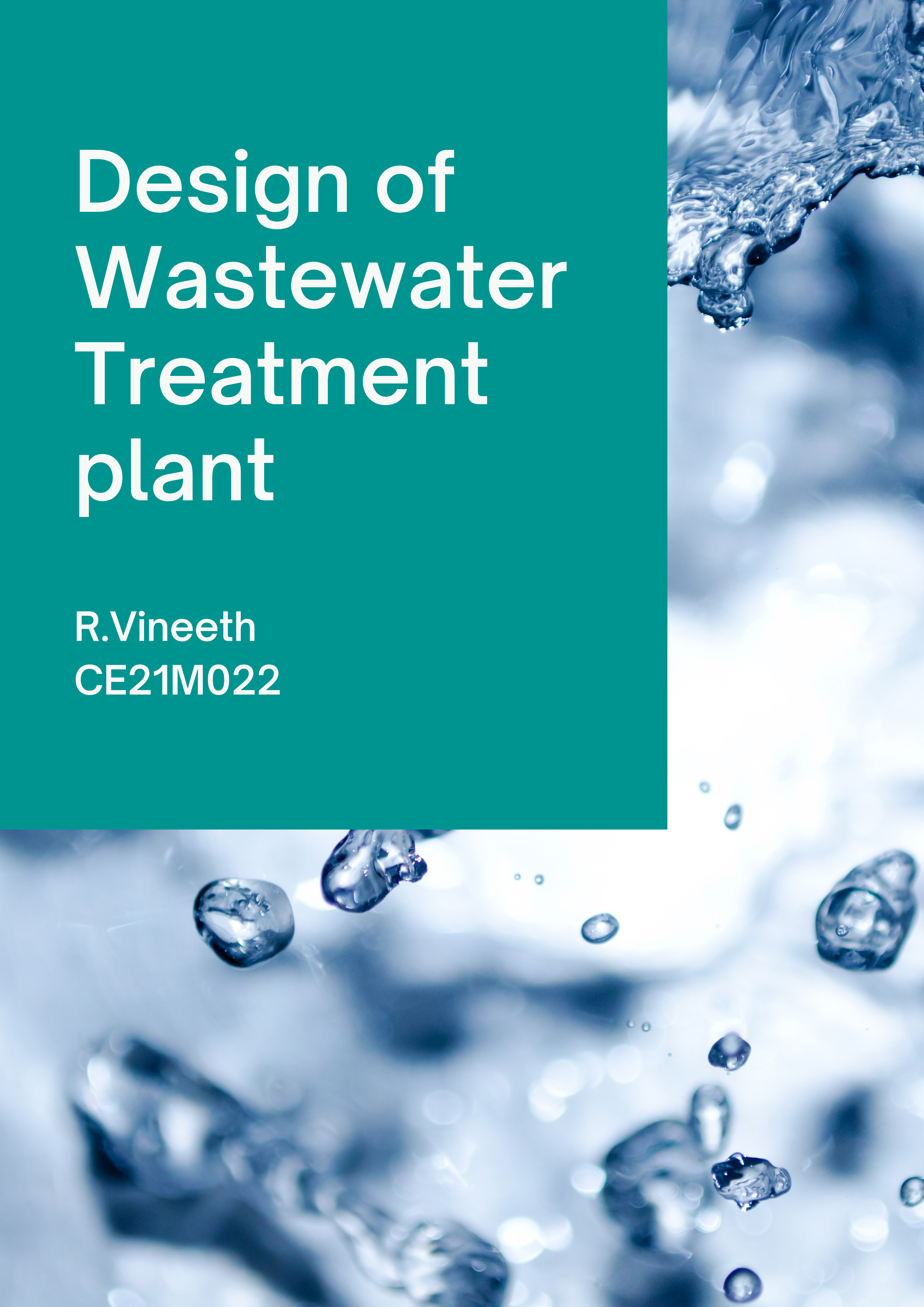
CE21M022



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Management System

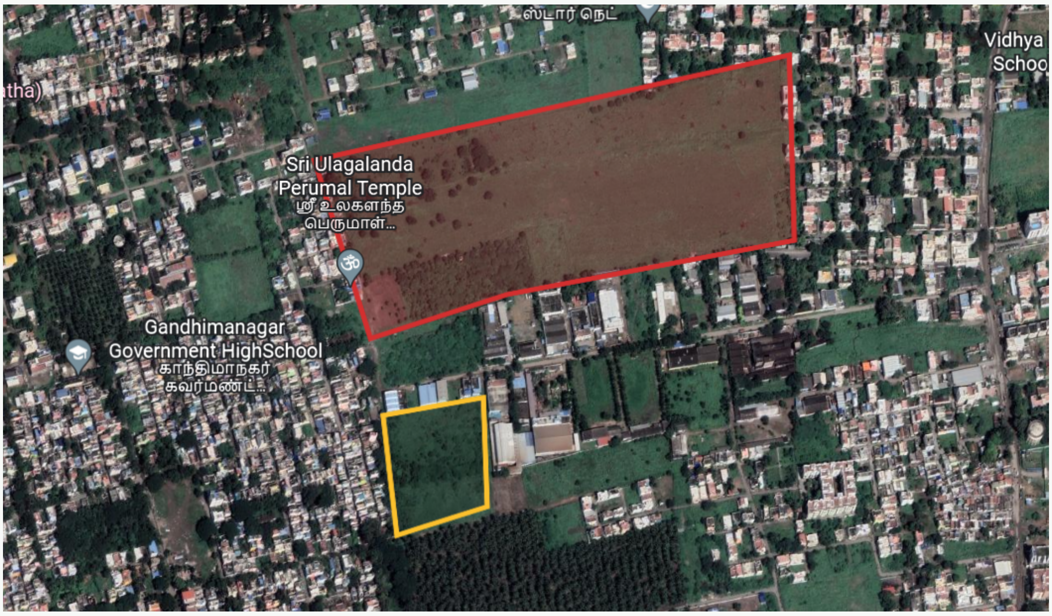
Laying down underground pipelines, transferring wastewater to a common point, is highly costly, corporations all around the world are looking for alternate ways to treat wastewater and enhance water sufficiency right from the local to regional level. Centralized system is not the one shot solution for every problem, decentralized systems are picking pace, so it's necessary to decide the type of management before getting on to the design. In order to take this decision, I had to rely on well established guidelines.

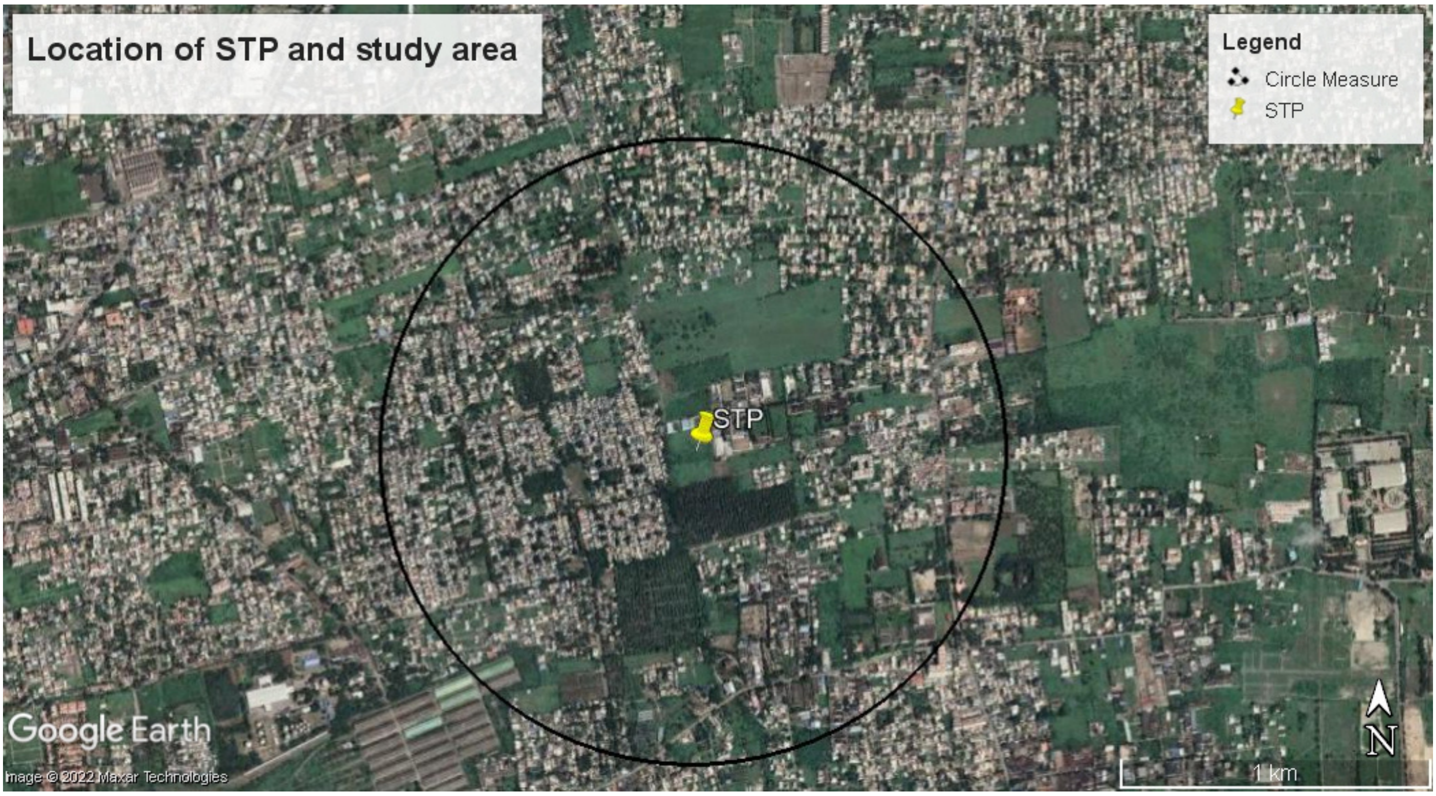
CPHEEO had provided guidelines for the selection of Management system, my locality has **well established underground sewage system**, already huge investments have gone into laying the pipelines, surprisingly two tracts of land in the centre of the area under consideration was available, if it had been decentralized, finding land to setup near the residential area would be difficult as houses are closely arranged, also getting approval from the neighbors for setting up an STP would be a tedious task, whereas the two **sites chosen for centralized is relatively away from residential area**, one site is very close to the industry, so public approval won't be an issue.

Coimbatore is not new to these issues, Nanjundapuram STP was stalled for 10 years as the residents filed a case demanding for stopping the construction work, same was the case with Ukkadam STP, CCMC had lost 15 crores due to the stoppage. It's necessary to plan the STP in such a way that it causes less trouble, else it will be embroiled in a long legal battle, so in my case **centralized wastewater management system would be ideal**, site chosen lies in the centre of the area. maximum length of the underground pipe would be 1 km. **In case if Sewage system of an entire city is being planned, it is better to prefer Decentralized system,** it would also bring down the cost of UGD.

CCMC HAD STARTED THE NANJUNDAPURAM STP CONSTRUCTION MORE THAN 10 YEARS AGO UNDER JNNURM, AFTER 8 YEAR LONG LEGAL BATTLE, IT HAS AGAIN RESUMED THE CONSTRUCTION OF STP BASED ON TRIBUNAL'S ORDER

Two options are available for setting up the centralized treatment facility, in order to select the best out of the two, topographical data was used.
Site with red outline had an average elevation of 426m, whereas the site with yellow boundary had an average elevation of 423m
Both the sites are almost at the centre of the area under consideration, so I have selected the site with the yellow boundary which has an elevation of 423m for the construction of sewage treatment facility





Land selected for setting up the STP has a total area of 23,400 sqm, I wanted to know if this piece of land is sufficient to contain a 7.511 MLD plant, so I needed a rough estimate of area required per MLD. I used google earth to find out the area of various STP and their treatment capacity.

Koyambedu sewage treatment plant occupies around 735 sqm per MLD
Perungudi Sewage treatment plant occupies around 618 sqm per MLD
IIT Madras Sewage Treatment Plant occupies around 450 sqm per MLD
These are very rough estimates, in 2051 the flow would be 7.511 MLD, so anything above 5,000sqm would suffice.

As the population is growing exponentially and then saturating, designing for 2051 flow alone won't be efficient, so design for 2031 flow is initially carried out, after this combined design of 2041 and 2051 is carried out, modules of grit, SBR, MGF, anaerobic digester and UF are added as the flow increases. **Modular approach** is more rational if the population is experiencing a geometric growth, in other cases when the population growth is stagnant, design of reactors for 2051 flow could be carried out.

Reuse Potential

The Sixth Assessment Report (AR6) Working Group II report — titled IPCC AR6 WGII 'Climate Change 2022: Impacts, Adaptation and Vulnerability' — has pointed out that changing climate, coupled with rising demand, could mean that about **40% of the people residing in India will live with water scarcity by 2050**, as compared to 33% of the population at present, according to a study cited by the report

Why Reclaimed water is best for industries?

In water stress situation the utility would prefer supplying water to the households over the industries, with many such droughts events stated to occur in the future due to climate change, this can jeopardize the operations of an industry.
In many cities reclaimed water is cheaper than standard piped water supply by utility.
Stricter regulations on groundwater extraction is also being levied. Singapore's success in using treated wastewater referred to as NEWater for industrial supply is a good case in point and is relevant to the discussion on water supply and reuse of treated wastewater in India

18,883 MLD
Sewage treatment capacity as of 2015

48 INR/KL
Weighted average of industrial water tariff

We possess technologies to polish the treated wastewater to meet the necessary industry guidelines. The National Water Policy (2012), National Urban Sanitation Policy (2008) have rallied for the reuse of wastewater in industries but there's lack of clear guidelines and framework to support the implementation of the project.

Supply side potential and Market potential for reuse has to be studied before implementation. Few industries might wish to use the treated wastewater for their main processes, in that case Grade 3 water has to be delivered.

404 Million Urban dwellers between 2015-2050

WATER DEMAND FROM HOUSEHOLDS, INDUSTRIES AND POWER PLANTS GROWING SIMULTANEOUSLY ADDING TO URBAN WASTER STRESS

Wastewater reuse projects are generally complex, it would require the expertise of Private players, so Public Private Partnership has to be forged for the successful operations. I will try to address this issue in the reuse potential section.

I started searching for potential clients in and around my locality.

- **BR Puram Industrial estate** houses industries that produce automobile parts and foundries
- **TIDEL** is a huge technology park situated 3km from the proposed STP, has a floor space of 23 lakh sq. ft currently employing 13,200 employees.
- **BPCL Gas bottling Plant** situated at a distance of 1 km from the STP
- **Prozone and Fun Mall** situated at a distance of around 1km from the STP
- **CHIL SEZ IT park** situated at a distance of 9 km from the STP, currently employing 20,000+ employees
- **Coimbatore International Airport** situated at a distance of 6km from the STP

In Bengaluru 10MLD tertiary treatment plant at Yelahanka supplies reclaimed water to Bengaluru International Airport, a similar model could also be adopted in the current site.
Now that potential clients are identified, it is important to bring them to a common table and forge a good contract. For flushing operations Grade I water is sufficient that is the treated water has to be subjected to Sand and carbon filtration. For it to be used as process water in many industries, then Grade III is needed, It has to be passed through Sand & Carbon filtration, Ultra/Micro Filtration and Reverse Osmosis.

Coimbatore International Airport, TIDEL, CHIL SEZ IT park, Fun and Prozone Mall would require Grade I water
BPCL Gas bottling plant and industries in BR Puram Industrial Estate would require Grade II water for uses like cooling.
We could estimate the quantity of Grade I water and Grade II water and setup a suitable Tertiary System.
Grade III would require substantial investment some kind of a Public Private has to be established. I came across a report published by Price Water House Cooper on closing the water loop: Reuse of Treated Wastewater in Urban India. One of the model which was proposed by them was,

Three Party Fixed Price Model (TPFP) :
Three players are involved namely, utility provider (City Corporation managing the STP), Technology provider cum contracter and industrial entity. They have to basically enter into a long term contract assuring supply of secondary treated water and reclaimed water at predetermined rates and quality levels.

THREE PARTY FIXED PRICE MODEL - A CONTRACT FORGED BETWEEN UTILITY PROVIDER, TECHNOLOGY PROVIDER AND INDUSTRIAL ENTITY

How does this contract work?
Technology provider would examine the secondary treated wastewater and then decide on the kind of tertiary system that would be needed, utility provider would provide land for the construction of tertiary plant. Technology provider could be the civil contractor also. Technology provider will supply the agreed quantity and quality of wastewater to the industrial units. Industrial units would pay volumetric charges to the utility provider at a predefined tariff rate.

With guaranteed water purchase from industrial units and other bulk consumers and a minimum guaranteed revenue from utility provider, there is reduced risk to Technology provider, as the risk reduces it would encourage the technology providers to involve themselves in such contracts.

We do have technologies to treat the water to maximum extent, it is the kind of relationship that we forge between various stakeholders, that will decide the successful implementation of the project. Various other contracts have also been discussed in the report starting from Reuse utility buy back model and End user reuse PPP.

Choice of Tertiary Systems:
As mentioned above the tertiary system would be implemented by Technology provider cum civil contractor (it could be any private player), but here I will try to elaborate on the possible treatment techniques that could be used. From the site visit to Nesapakkam STP, I got an overview on the tertiary systems employed, as this treated water is sent back to Porur lake, which is also houses water intake structure, so maximum possible treatment in provided. Multi grade filter, UF and Ozone were the tertiary treatment technologies used, so adopting the same convention. In this STP, the Technology provider could Implement MGF and UF as the tertiary systems.

Reclaimed water could be taken to the potential clients as mentioned in the previous page. Reclaimed water could be transported via lorries or constructing UGD's.

If we could add in another module of Reverse Osmosis, it is termed as Grade III water and can be used as process water in many industries.

WE POSSESS TECHNOLOGIES THAT CAN TREAT THE EFFLUENT FROM STP TO HIGHEST QUALITY BUT IT'S THE RELATIONSHIP WE FORGE BETWEEN VARIOUS STAKEHOLDERS THAT DETERMINES THE SUCCESS OF THE PROJECT

Population Forecasting

My Native place falls under Vilankurichi region of Coimbatore and It's located within the corporation limits. 1 km radius was chosen from the centre of the STP site. From the population data it was found that there was a rapid increase in the population, It has a coverage of 9.2 km^2

Year	Population	Growth Percentage	Population within 1 km radius from the STP Site
1981	2690		919
1991	5130	90.70%	1751
2001	9124	77.85%	3115
2011	24235	165.62%	8272

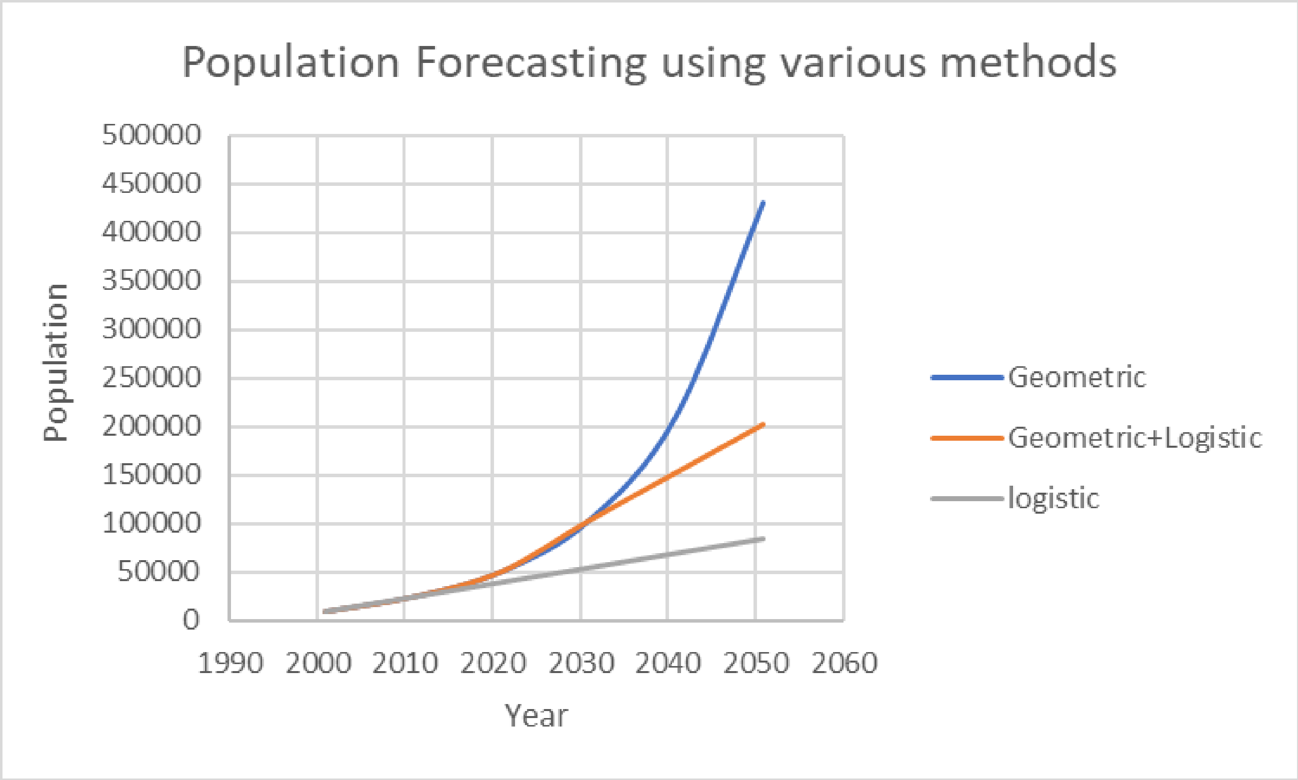
I have assumed the saturation population to be 2,00,000 (which corresponds to a density of 21,700/km^2 , which is at par with Chennai).

I have posted the comparison between geometric method and logistic growth method, In 2021 and 2031 estimate from logistic model is very less, given that this area is very rapidly developing, to prevent underestimation of population, I had used geometric method for 2021 and 2031, using that data with saturation population as 2,00,000 I found the population in 2041 and 2051. Logic behind doing such analysis is flow should not be underestimated and exponential growth over long periods can't take place, population will reach saturation due to various constraints.

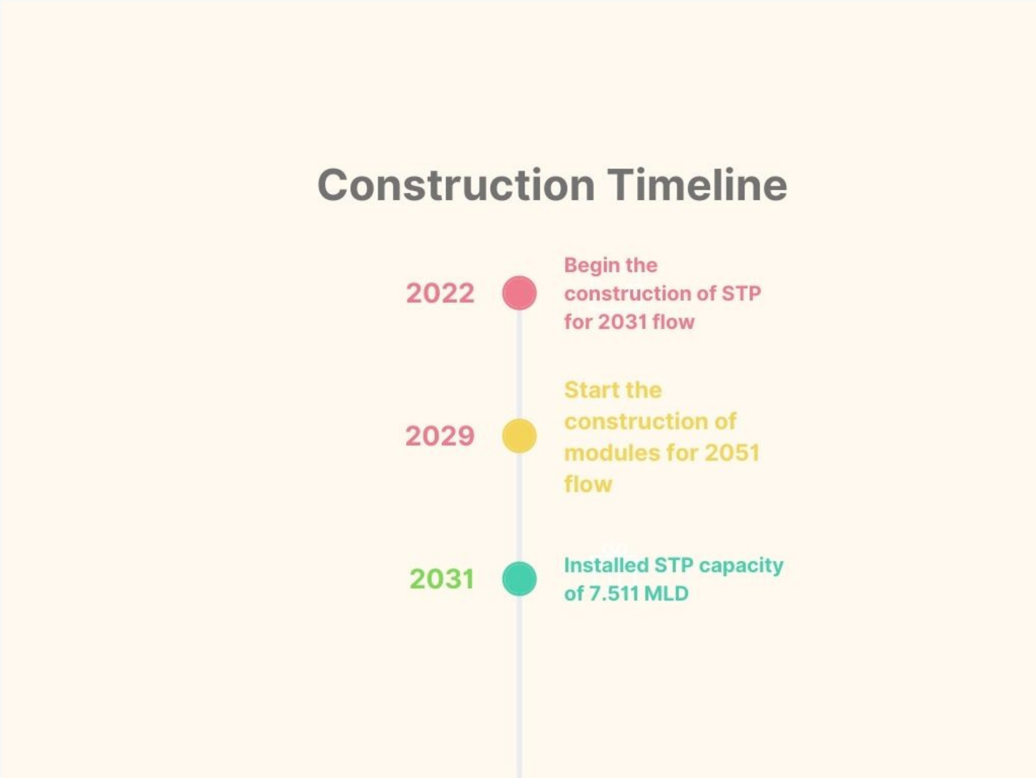
Year	Population forecasting using geometric method	Population forecasting by Logistic growth model
2021	49770	39346
2031	102207	54457
2041	209892	69568
2051	431035	84679

POPULATION FORECASTING USING A COMBINATION OF GEOMETRIC AND LOGISTIC GROWTH MODEL

Year	Population	Population within 1 km radius from the STP Site
2022	57445	19616
2031	102207	34884
2041	151942	51884
2051	201677	68868



Using Geometric method alone would have been huge overestimation and that is not practical also. Only using logistic method could have been an underestimate, so a combination of geometric and logistic method (which takes into account the saturation population) was used in this report



Wastewater Quantification

Population within 1 km radius was found out using geometric increase method, Next step was to find out the schools, hospitals, offices and restaurants within the 1 km radius to quantify the water requirement.

Entity	CPHEEO Recommendation	Water requirement
Gandhimaanagar Government school	45lpcd	10350 L/d ((230 staff and students)*45)
SS Hopsital	340 Litre per bed	6800 L/d (20 Bed hospital*340)
Vidya Nikethan School CBSE	45lpcd	11250 L/d ((250 Staff and students)*45)
Shri Raman Chettiar Memorial School	45lpcd	15750 L/d ((350 Staff and students)*45)
MJ Vincent Matriculation School	45	9000 L/d ((200 Staff and students)*45)
Reshmi Industries	45lpcd	6750 L/d ((150 staff*45))
Harini Hospital	340 per bed	3400 L/d (10 bed hospital)
Restaurants	70 per seat	10500 L/d (10 restaurants with an average seating of 15)

Wastewater Quantification

Population within 1 km radius was found out using geometric increase method, Next step was to find out the schools, hospitals, offices and restaurants within the 1 km radius to quantify the water requirement.

2022

- Residential Water demand = 2648160 L/d (19616*135)
- Schools, offices and restaurants = 73800 L/d
- Total Demand = 2.17 MLD (80% of total)

2031

- Residential Water demand = 4709340 L/d
- Schools, offices and restaurants = 79335 L/d (7.5% growth)
- Total Demand = 3.83 MLD

2041

- Residential Water Demand = 7004340 L/d
- Schools, offices and restaurants = 85285 L/d
- Total Demand = 5.67 MLD

2051

- Residential Water Demand = 9297180 L/d
- Schools, offices and restaurants = 91618.5 L/d
- Total Demand = 7.511 MLD

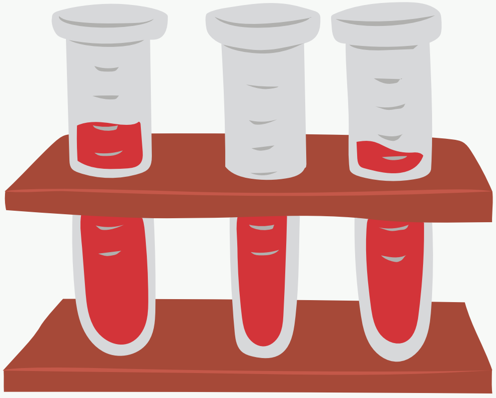
Characteristics of Wastewater

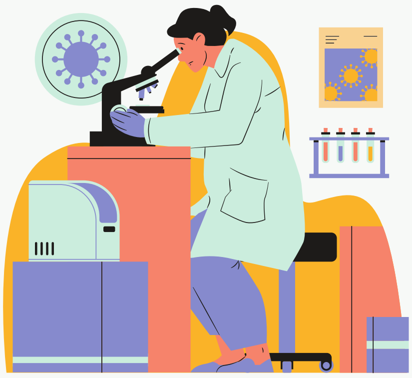
A study was conducted on the functioning of sewage treatment plants in Coimbatore by Tamil Nadu Agricultural University. One of the sewage treatment plants under consideration was the Ukkadam STP, so the values I have listed here is from the paper published by those researchers. I am listing down few important characteristics of wastewater

Parameters	Pre Monsoon	Post Monsoon
BOD (mg/l)	315-324	249-260
COD (mg/l)	740-779	531-532
TSS (mg/l)	346-353	590-609
TDS (mg/l)	1551-1556	1537-1542
pH	7.56-7.67	7.35-7.46
TKN (mg/l)	34-35	34-35
Cl- (mg/l)	68-69	41-42
NH4-N (mg/l)	5.5-5.6	4.2-4.3

Characteristics of Wastewater

Parameters	Pre Monsoon	Post Monsoon
Nitrate (mg/l)	0.44-0.45	0.24-0.25
Phosphate (mg/l)	4.9-5.1	4.9-5.1
Total Alkalinity (mg/l) as CaCO3	81-83	73-75
Bacteria x10^6 CFU/ml	26-27	44-45
Electrical Conductivity dSm^-1	2.91-2.92	2.54-2.55
Sulphate (mg/l)	67-69	70-72





Selection of Treatment Technology

Challenges in wastewater management is to choose the appropriate technology for a particular wastewater treatment objective at a particular site. Factors like Capital Cost, Operation and Maintenance, Land requirement will influence the final decision.

Lot of research has gone in addressing these issues, as multiple criteria has to be evaluated for various alternatives. MCDM (Multi criteria decision making) has been extensively used in operations research and environmental management to identify the appropriate technology.

A study from IIT Bombay had used MADM (Multi attribute decision making) which is a subset of MCDM to identify the best biological treatment under various scenarios and criteria.

IN THE CURRENT TIMES DECISION MAKING FRAMEWORKS SHOULD INCORPORATE SUSTAINABILITY INDICATORS

TOPSIS (Technique for order of preference by similarity to ideal solution) is one of the numerical method to solve MCDM problems. Biological wastewater treatment technologies that were compared for the study include Conventional Activated sludge process, UASB followed by facultative aerated lagoon, SBR and constructed wetlands.
Criteria that was selected in the study include,

1. Global Warming
2. Eutrophication
3. Life cycle costs
4. Land requirement
5. Manpower requirement for operation
6. Robustness of the system
   6.1 Reliability
   6.2 Durability
   6.3 Flexibility
7. Sustainability
   7.1 Acceptability
   7.2 Participation
   7.3 Replicability
   7.4 Promotion of Sustainable behavior

First five criteria was obtained from LCA and LCC studies, last two criteria had one or more indicators (attributes) was obtained by interacting with professionals. To add layers on top of it, scenarios were created to emphasize that appropriate technology would vary depending upon the site and various constraints in that locality. These scenarios gave weights to the criteria and the treatment technologies were ranked.
Reliability points to the possibility of achieving adequate performance for a specific period of time
Durability is defined as the technological life time.
Flexibility is the ease with which the plant could be upgraded to take in more organic or hydraulic load.
People from different culture have different perception on waste and sanitation, this aspect is taken care by **acceptability**.
Technology being implemented should promote public participation and make the community responsible for the success of the implementation of the project, this aspect is taken care by the indicator **public participation.**
Replicability indicator captures the simple design, implementation and operational features of the technology.

TOPSIS FINDS THE DISTANCE BETWEEN AN ALTERNATIVE AND IDEAL AND NON-IDEAL POINTS FOR A GIVEN CRITERIA, USING WHICH CLOSENESS FACTOR IS ESTIMATED AND ALTERNATIVES ARE RANKED FOR A CRITERIA

Scenarios chosen for the study include,

1. Urban area/Land constraint/disposal to surface water body
2. Urban area/Land Constraint/ Treated water for reuse
3. Sub-Urban area /no land constraint/ disposal to surface water body
4. Sub-Urban area/ no land constraint/treated water reuse

two other similar scenarios for rural area also was considered in the study. In my case Scenario 2 perfectly defines my locality and reuse potential. With land prices shooting upwards, it is always better to choose technology that occupies less area. To give actual estimates, 1 cent of land in my locality costs upward of 20 lakhs, coupled with this is the exponential growth in population, so land is one of the most important criteria.
Closeness ratio was estimated for alternatives in various scenarios and they were ranked.

For scenario 2 it was found that SBR had a rank of 1 followed by conventional ASP, UASB with facultative aerated lagoon and constructed wetlands.
SBR has the least footprint and the quality of effluent is also very good, when it comes to robustness of the system and sustainability both ASP and SBR perform equally, it's in land requirement and eutrophication potential SBR beats ASP.

Site I have chosen is inside the corporation limits and it's stated to develop with many tech parks and TIDEL park starting operations in saravanampatty.
It also houses many foundries and steel godowns of SAIL and Vizag steel and an Industrial estate
It is logical to provide more weightage to land footprint in choosing the best alternative, given that there's an exponential growth in population.
Economic and eutrophication potential of the wastewater makes SBR a better solution than ASP.

Selection of wastewater technology is very difficult problem, various criteria has to be considered, which should satisfy the stakeholders involved in the problem, MADM approaches this problem in a very scientific way by quantifying various criteria and adding mathematical formulation over it.

FOR THE SCENARIO, URBAN AREA WITH LAND CONSTRAINT AND REUSE OF TREATED WASTEWATER SBR SEEMS TO BE THE BEST OPTION

Another question which might arise out of this decision is, why didn't I consider Modified version of ASP like Extended aeration, ASP combined with biological nutrient removal, above mentioned technologies produce more or less same quality effluent, approximately have same treatment plant footprint and cost per MLD as ASP. It's strongly reinforced that SBR is better in an urban setting with land constraints and reuse potential. Another question which might arise is, even MBR has less footprint and produces high quality effluent.

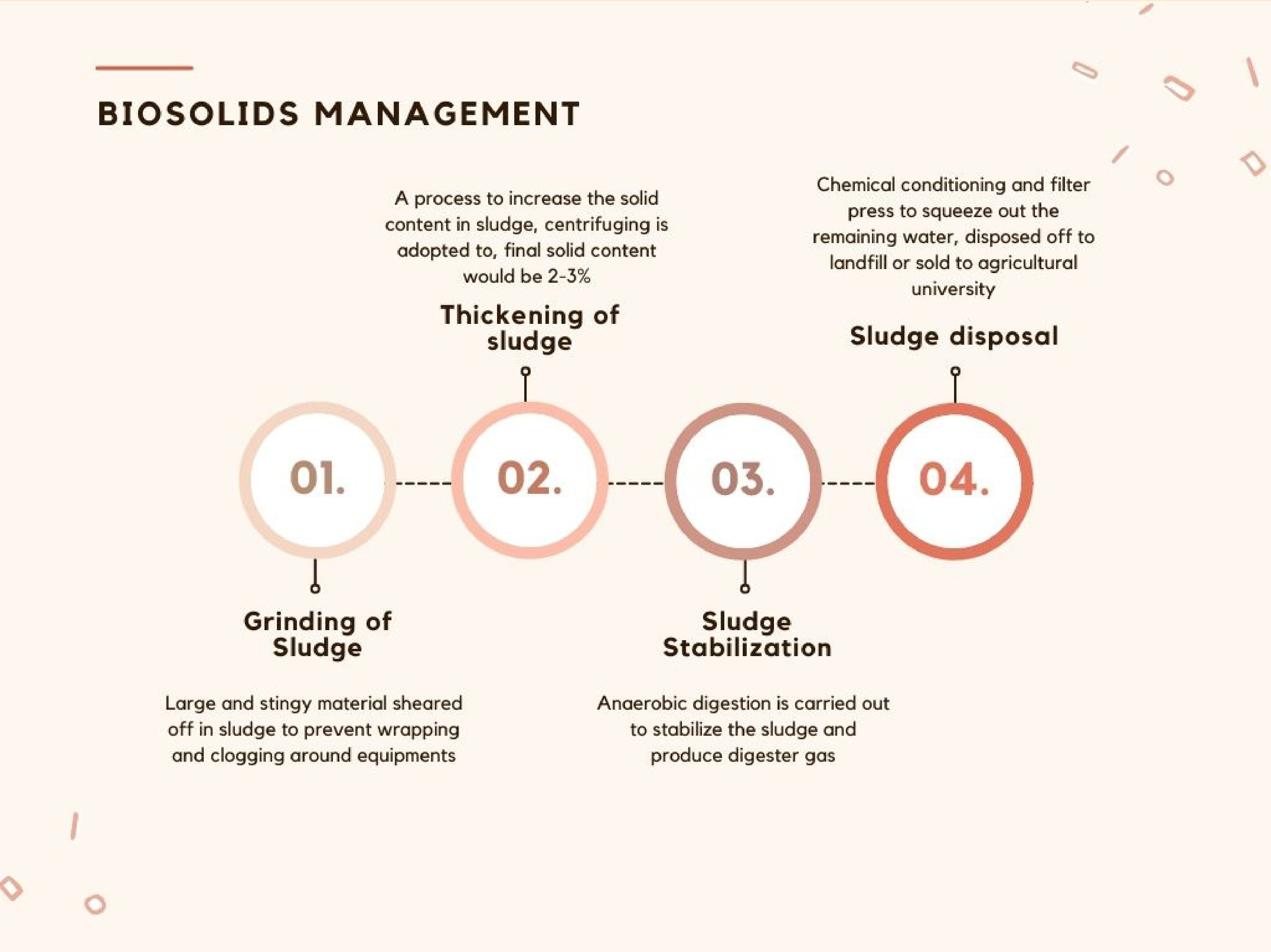
Why not MBR over SBR?
Yes MBR produces high quality effluent and it occupies less space, which is the ideal criteria in an urban setting. I read a report prepared by experts from various IIT's for Ganga River Basin: Environmental Management Plan. They simultaneously compared various treatment alternatives under various criteria. This report was published in 2010, but it could be taken as an indication for selecting the best alternative. Total capital cost including both the secondary and tertiary systems for an SBR was around 115 lakhs per MLD, for MBR it was 300 lakhs per MLD (2010 estimates), there's a significant difference in the estimates, area requirement for SBR is 100 m^2 greater than MBR area requirement. When it comes to yearly power cost also money spent on MBR is twice that of SBR.

Another reason for not choosing MBR over SBR is the treatment cost per paisa, If this goes up, then the end consumers of treated wastewater would be discouraged to purchase it from the utilities, It is necessary to keep this cost down, to reinforce a robust contract between the utility provider, Technology provider and end user.

Taking all of these factors into consideration, it's proven that in an **Urban setting with land constraint and treated water being subjected to reuse SBR is best.**

IT IS NECESSARY TO BRING DOWN THE TREATMENT COST/L TO FORGE A ROBUST CONTRACT BETWEEN THE STAKEHOLDERS

FLOW CHART OF BIOSOLIDS MANAGEMENT



Approach Channel

It is a rectangular channel, that transports the wastewater from the Raw sewage collection sump to the screens and the subsequent downstream processes. It transports raw sewage via gravity flow and it's necessary to maintain a minimum velocity of 0.45 m/s, this can be checked using manning's equation.

Design Flow = 7.511MLD (for a population of (147115)

Peak Factor = 2.25

Maximum flow = 16.89MLD (0.195 m^3/s)

Assuming a flow through velocity of 0.6 m/s which is greater than 0.45 m/s

Required cross sectional area is = 0.325 m^2 (0.195 m^3/s / 0.6 m/s)

Let's provide 1 channel, with **Breadth to depth ratio as 1.5:1**

Breadth of the channel = 0.46m (say 0.5m)

Depth of the channel = 0.75m (say 0.75m), providing as free board of 0.5m

Provided cross sectional area = 0.375m^2

It was mentioned in CPHEEO that a certain section of **approach channel had to be straight** with a **length of at least 5 times the width of the channel**

So, **Length of the channel** = 2.5m

Check:

Using manning's equation to find if the design is sufficient

When **it's flowing full** Hydraulic radius would be,

R = 0.1875 (0.375m^2/ ((2*0.75) + 0.5))

Manning's coefficient for concrete = 0.015 and let the slope be 0.001C

Velocity from manning's equation is = **0.69 m/s > 0.45 m/s**

When **it's flowing at depth of D/3** then the hydraulic radius would be,

R =0.125 (0.125m^2/ (1m))

Velocity from manning's equation = **0.527 m/s > 0.45 m/s**

Hence the given design can be implemented

Summary:

Rectangular channel made with concrete with a slope of 1 in 1000, length of the straight section being 2.5m and breadth and depth of the channel are 0.5m and 0.75m (extra 0.5m as free board respectively

Screening

Screens are used for removing huge suspended particles from the wastewater stream, so that these substances don't affect the downstream machinery and the process. There are different type of screens available depending on the size of opening namely, coarse, fine and medium screens. Fine screens are generally not used for sewage, as it is susceptible to clogging.

Screens with intermittent cleaning arrangement are likely to produce surges of relatively high flow soon after cleaning, this is not the case with mechanical screening, where it is continuously cleaned.

Design aspect – Choose a velocity that is not very low that it allows settling of particles in the channel and it should not be very high, so that it does not interfere with the screening process. Ideal velocity – (0.6 m/s – 1.2 m/s).

Depth of the screen = 0.75m

Width of the screen = 0.5m (equal to the width of the approach channel)

Approach velocity = 0.52m/s

Providing 10mm dia bars at a clear spacing of 25mm, **No. of bars required would be**,

N = (700/ (25+10)) +1, which is equal to **16 bars**

Since the bars are obstructing the flow, actual width of flow would change also bars would be placed at an inclination of 45^o

Area of the screen when inclined at 45^o = $0.4407m^2$

Effective area of the screens including the openings = $0.314m^2$ (0.4407*((25/1000)/((25/1000) + (10/1000))))

Velocity through the screen = 0.621 m/s

Head loss = 10.6mm <150mm ($0.0729(0.621^2 – 0.49^2)$), hence the design is correct

Summary:

A Screen made up of 10mm dia bars at 25mm spacing (16 Nos) with depth and width of the screen as 0.75m and 0.5m respectively, inclined at an angle of 45^o

Grit Chamber

Horizontal flow through velocity in grit chamber is decided based upon the ability to not scour the already settled particles. Critical velocity can be given by the following equation (Rao and Dutta 2007)

$$Vc = \sqrt{\left[\frac{8\beta}{f} g(S-1)D\right]}$$

β, f and S are properties of grit, objective of grit chamber is to remove smallest particles of size 0.2mm with a specific gravity of 2.65

When these conditions are substituted in the above equation, scour velocity turns out to be 0.228 m/s, so if we assume horizontal flow through velocity, it should be less than 0.228 m/s

Sewage generated (**2031 flow**) = **0.044 m^3/s**

Maximum flow = **0.132 m^3/s** (Peak factor is 3 for population less than 20,000)

Minimum temperature in Coimbatore is 21^oC, finding out the settling velocity at this temperature, as it would be minimum in this case, designing for the worst-case scenario

Kinematic viscosity at 21^oC = 0.9795*10^{-2} cm^2/s

Let's initially assume that Reynolds number is less than 0.3

Estimating the settling velocity using stokes law

$$Vs = \frac{g}{18}\left[\frac{S-1}{\nu}\right]D^2$$

Velocity in the first trial = 0.036 m/s

Reynold's number = 7.498 which is greater than 0.3, hence the flow is not laminar

It's transitional flow

After performing various trials velocity turns out be = **0.026m/s (Settling velocity)**

Scour velocity = **0.228 m/s**

Keeping the horizontal flow velocity less than 0.228 m/s, so the Horizontal velocity assumed is = **0.2m/s**

Cross sectional area = 0.66 m^2 (0.132 m^3/s)/(0.2 m/s)

Providing the width of the grit chamber as 1m, then the depth of the grit chamber would be 0.66m (say 0.7m)

V_o/V_c = H/L, using this relationship, Length of the tank is estimated

$$L=(H*V_C)/V_O$$

Length of the grit chamber = 6.13m

Increasing the length by 25% to accommodate the inlet and outlet zones, length of grit chamber = 7.67 m (providing 8m)

Working volume = 5.6 m^3

Providing allowances for free board and grit accumulation zone,

Total depth = 1.25 m

Detention time = 42.42 sec

Design summary:

1 Grit chamber with Length = 8m, Breadth = 1m and depth = 1.25m with horizontal flow through velocity as 0.2m/s and detention time of 43 sec

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Sewage generated (**2051 flow**) = **0.043 m^3/s (difference between 2051 and 2031 flow)**

Maximum flow = **0.0965 m^3/s** (Peak factor is 2.25 for population less than 50,000-750000)

Minimum temperature in Coimbatore is 21°C, finding out the settling velocity at this temperature, as it would be minimum in this case, designing for the worst-case scenario

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It's transitional flow

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Scour velocity = **0.228 m/s**

Keeping the horizontal flow velocity less than 0.228 m/s, so the Horizontal velocity assumed is = **0.228m/s**

Cross sectional area = 0.423 m^2 (0.0965 m^3/s)/(0.228 m/s)

Providing the width of the grit chamber as 1m, then the depth of the grit chamber would be 0.423m (say 0.5m)

V_o/V_c = H/L, using this relationship, Length of the tank is estimated

$$L=(H*V_C)/V_O$$

Length of the grit chamber = 4.384m

Increasing the length by 25% to accommodate the inlet and outlet zones, length of grit chamber = 5.48 m (providing 5.5m)

Working volume = 2.75 m^3

Providing allowances for free board and grit accumulation zone,

Total depth = 1.05 m (Providing 1.2m)

Detention time = 26.44 sec

Design summary:

1 Grit chamber with Length = 5.5m, Breadth = 1m and depth = 1.2m with horizontal flow through velocity as 0.228m/s and detention time of 26.44 sec

Velocity control devices for Grit chamber

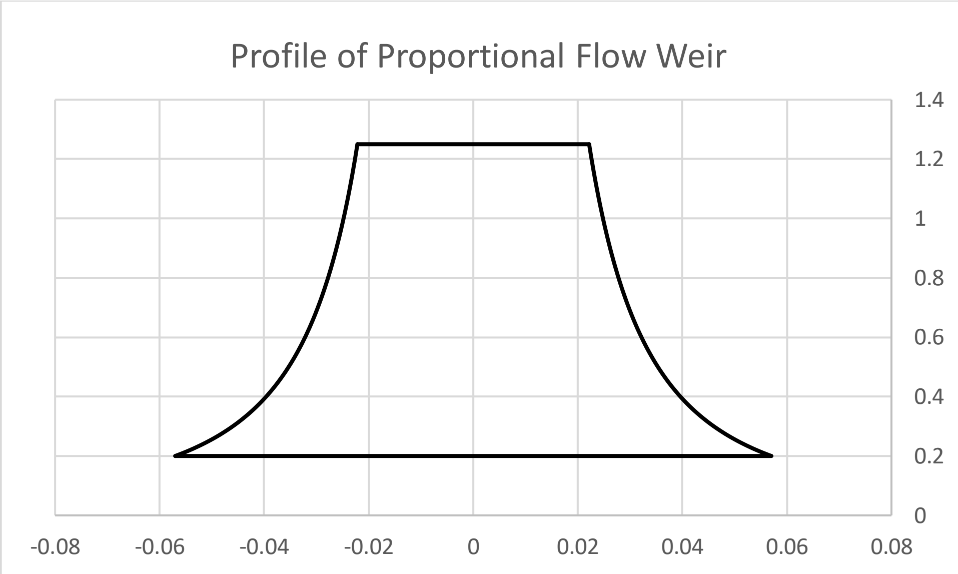
It is necessary to maintain a constant velocity in the grit chamber, this is done by placing a proportional weir in the outlet which maintains a constant velocity in the grit chamber by varying the cross-sectional area of flow through the weir so that the depth is proportional to the flow.

CPHEEO has given the equation for the designing the profile of the proportional weir

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

X is the weir width at the liquid surface and y is the depth of flow, a is a constant that varies between 25mm and 50mm and b is base width if the weir

Substituting the values in the above equation, gives the profile of the proportional weir as



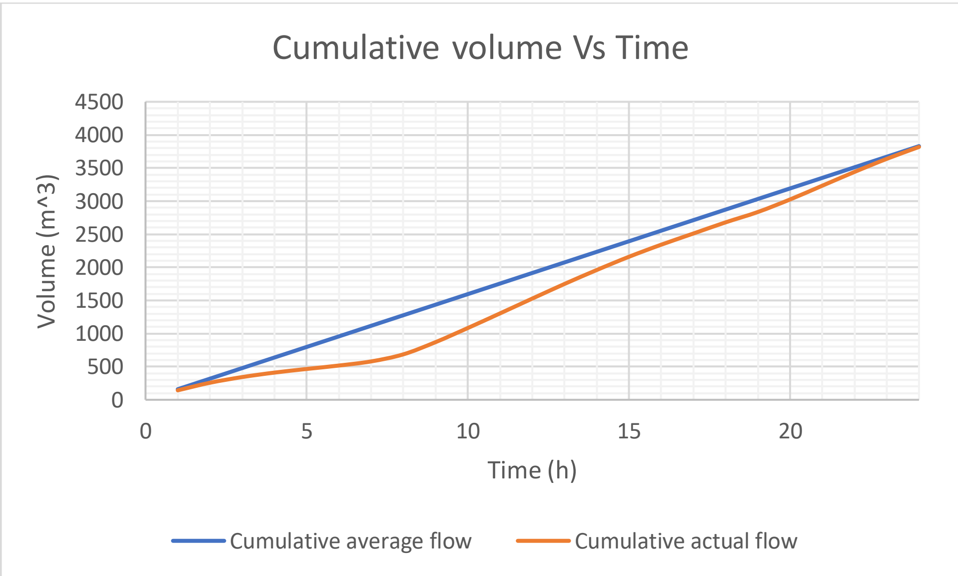
The weir shall be set from 100-300mm from the bottom of grit chamber to provide grit storage of for operation of mechanical grit cleaning.

Equalization Tank

Design flow rate (2031) = 3.83 MLD

Average flow per hour = 159.5 m^3/hr

Data was available for 24 hrs, graph was plotted between cumulative volume and time, cumulative average flow and cumulative actual flow was plotted and the maximum difference was found out, which is basically the volume of the equalization tank.



Maximum difference = 590.5 m^3, this is also the volume of equalization tank. Increasing the volume by 15% to account for the free board and other factors

Volume of the Equalization Tank = 680 m^3

Providing **one equalization tank with a depth of 4m**, another 0.5m for free board.

Surface area of Equalization tank = 151.11 m^2

Providing a rectangular tank with L: B as 1.5:1

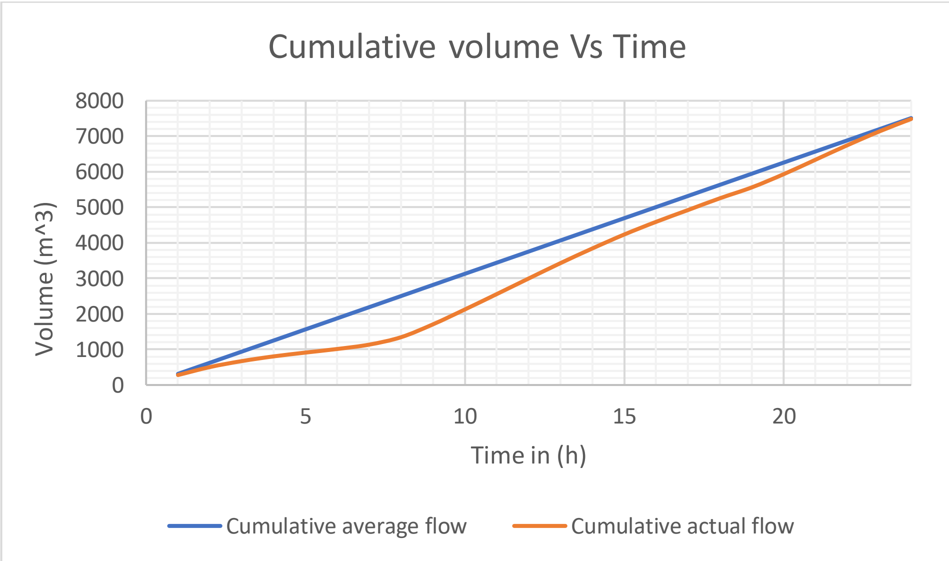
Length of equalization tank = 15.5m, **Breadth of Equalization Tank** = 10.5m

Equalization Tank

Design flow rate (2051) = 7.511 MLD

Average flow per hour = 312.95 m^3/hr

Data was available for 24 hrs, graph was plotted between cumulative volume and time, cumulative average flow and cumulative actual flow was plotted and the maximum difference was found out, which is basically the volume of the equalization tank.



Maximum difference = 1158.02 m^3, this is also the volume of equalization tank. Increasing the volume by 15% to account for the free board and other factors

Volume of the Equalization Tank = 1331.72 m^3

Already installed capacity of 680 m^3 so the extra module that has to be fitted would have a volume of 652 m^3

Providing **one equalization tank with a depth of 4m**, another 0.5m for free board.

Surface area of Equalization tank = 163 m^2

Providing a rectangular tank with L: B as 1.5:1

Length of equalization tank = 16m, **Breadth of Equalization Tank** = 10.5m

Design Summary

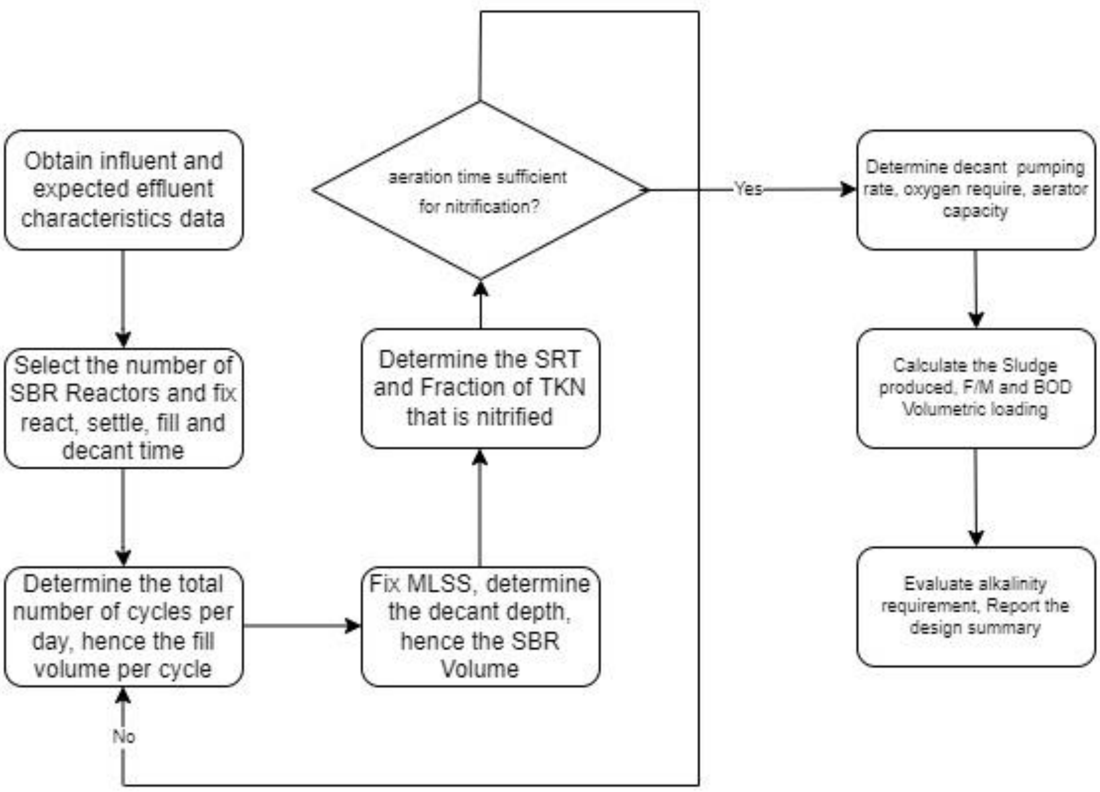
2031 flow Equalization tank:

- One tank with Length (15.5m) and breadth (10.5m)

2041 and 2051 combined flow Equalization tank:

- One tank with Length (16m) and breadth (10.5m)

Design of Biological treatment system (SBR)



Influent characteristics:

Constituent	Concentration (mg/L)
BOD	320
Soluble BOD	128
COD	760
Soluble COD	304
Readily Biodegradable COD	130
TSS	350
VSS	280
TKN	35
NH_4-N	5.6
TP	5
Alkalinity	82

Assumptions involved in the design of SBR

1. 2 Tanks are used
2. Total liquid depth when full – 4m
3. Decant depth – 20% of tank depth
4. Sludge volume index – 150 mL/g
5. Biodegradable COD = 1.6*BOD
6. MLSS is assumed to be 3500 mg/L

Biodegradable COD = 1.6*320

= 512 mg/L

Biodegradable Soluble COD = 1.6*(Soluble BOD)

= 1.6*128

= 204.8 mg/L

Non-biodegradable soluble COD = Soluble COD – Biodegradable soluble COD

= 99.2 mg/L (304-204.8)

Non-biodegradable particulate COD = COD-bCOD-Non biodegradable soluble COD

= 148.8 mg/L (760-512-99.2)

Finding out grams of COD per gram of Volatile Suspended Solids = (COD-sCOD)/(VSS)

= 1.628 g of COD/ g of VSS

Non-biodegradable Volatile Suspended Solids = 91.4 mg/L (148.8/1.628) (Numerator is non-biodegradable particulate COD)

Finding out SBR operating cycle

Different stages in SBR include

1. Fill (t_f)
2. React (t_r)
3. Settle (t_s)
4. Decant (t_d)
5. Idle

In this step, I will be assuming the durations for each step

1. t_r = 1 h
2. t_s = 0.5 h
3. t_d = 0.5 h

Fill time would be the summation of react, settle and decant period, since I am using two SBR then the total cycle would be twice the fill time

t_f = 2 h (1+0.5+0.5)

Total cycle time = 4 h (2*t_f)

Number of cycles per day per reactor = 6 (24/4)

Total cycles combined of two reactors = 12

Fill volume per cycle = (3830 m^3/d)/ (12)

= 319.17 m^3/ fill

Tank depth is assumed to be – 4m

Decant depth = 0.8 m (4*0.2)

Volume of the Tank = 1595.85 m^3 (319.17/0.2)

Hydraulic retention time = 20 h (2*1595.85 m^3 *24 h)/ (3830 m^3/day)

Actual sludge retention time, which was calculated using the formula provided in Metcalf and eddy (Pg 777) yielded 10.1 days, so **assuming the sludge retention time to be 10.5 days.**

Determining the MLVSS concentration

Using the same formula that was used for finding out SRT, can be used here with SRT as 10.5 days, which is equated to MLVSS * Volume of Tank

Taking out few important values from Table 8-14 of Metcalf and Eddy

Yield = 0.45 g of VSS/ g of bCOD

Endogenous decay coefficient = 0.12 (has to be modified for Average temperature of 29°C)

= 0.17 ($0.12*(1.04^{(29-20)})$)

Yield of Nitrifiers = 0.15 g of VSS/ g of NO_X

Endogenous decay of Nitrifiers varies in aerobic and anoxic conditions, so a weighted average is preferred. During the fill time, the reactor is in aerobic condition, once the fill stage is completed nitrifiers would get into action and ammonia would be converted to nitrate, so anoxic condition is established.

Fraction of fill time with respect to the entire cycle time is – 0.5 h (2 h/ 4 h)

Endogenous decay of nitrifiers in aerobic environment – 0.17 g/gd

Endogenous decay of nitrifiers in anoxic environment – 0.07g/gd

Average Endogenous decay =<u>0.155</u> ($0.17*(1.029^{(29-20)})*(0.5) + 0.07*(1.029^{(29-20)})*(1-0.5)$)

Fraction of cell mass remaining as cell debris = 0.15g/g

An expression for total solids wasted in terms of Volatile suspended solids is provided in Metcalf and Eddy denoted as $P_{X,VSS}$ (g of VSS/d)

Now that we have the value for SRT, ($P_{x,VSS}$ * SRT) would yield Total MLVSS (g) in the aeration tank

Equation for $P_{x,VSS}$

Would be contributed by Heterotrophic biomass, Cell debris and Non-biodegradable VSS in effluent

$$= \frac{Qy_H(s_0-s)SRT}{1+b_H(SRT)} + Q * nbVSS(SRT) + \frac{QY_n * NO_X(SRT)}{1+b_n(SRT)} + \frac{f_d b_H QY_H * (S_0-S)SRT^2}{1+b_H(SRT)}$$

Substituting the necessary data into the equation (SRT taken as 10.5 days) yields

$P_{x,VSS}$ = 4062185.26 g

MLVSS (g/m^3) = 2545.5 g/m^3 (mg/L) (4062185.26/1595.85)

MLVSS/MLSS = 0.727 (2545.5 mg/L /3500 mg/L)

Next step would be to identify the nitrogen that is oxidized which is given as

Nitrogen in the influent – Nitrogen in the effluent – Nitrogen in cell mass

First step would be to identify the total cell mass generated per day which is given by $P_{x,bio}$

$$\frac{Qy_H(s_0-s)}{1+b_H(SRT)} + \frac{QY_n * NO_X}{1+b_n(SRT)} + \frac{f_d b_H QY_H * (S_0-S)SRT}{1+b_H(SRT)}$$

Cell mass produced per day = 204356.3 g/day

= 204.35 kg/day

Inflow TKN is 35 mg/L

Effluent nitrogen required is 1 mg/L

Amount of Nitrogen oxidized = 21.194 mg/L (35-1-((0.12*204356.3)/ (1595.8)))

The task would be to find if the Level of nitrification reach 1 mg/L in the given span of 1h for aeration

Oxidizable NH_4-N added per cycle = (319.17*21.194*1000)

= 6764540.2 mg/fill

NH_4-N remaining in the tank before the fill = 1*(1595.8-319.17) *1000

= 1276666.67 mg/fill

Total oxidizable N at the Beginning of the cycle = (6764540.2+1276666.67) mg/fill

= 8041206.87 mg/fill

Initial concentration = 5.03 mg/L (8041206.87/ (1595.8*1000))

We need to know the quantity of nitrifiers in the system that would perform the oxidation of ammonium nitrogen to nitrate and nitrite nitrogen

$$\frac{QY_n * NO_X(SRT)}{1 + b_n(SRT)}$$

Nitrifier concentration = 17.49 mg/L

Kinetic co-efficient for nitrifiers were extracted from Table 8-14 of Metcalf and Eddy

$\mu_{m,\,29}$ =1.682 d^{-1} (0.9*$(1.072)^{29-20}$), $K_{NH4\;29}$ = 0.934 g/m^3 (0.5*$(1.072)^{(29-20)}$), K_o = 0.5 g/m^3

$$k_{NH_4} \cdot \ln\left(\frac{N_0}{Nt}\right) + (N_0 - N_t) = x_n\left(\frac{\mu_{m_{ax}}}{y_n}\right)\left(\frac{s_0}{k_0 + s_0}\right)t$$

N_0 = 5.03 mg/L, N_t = 1 mg/L, S_0 is the minimum dissolved oxygen in the wastewater

S_0 = 2 mg/L

Finding t from the equation is the objective and we have to check if that value is less than the aeration time, if it's less then we have to proceed with finding the oxygen requirement and aeration design.

After substituting the values, t from the above equation is 0.848 h which is less than the aeration time of 1 h hence there is sufficient time to oxidize the incoming nitrogen to the prescribed standards.

Decant pumping rate = Fill volume per cycle / decant time

= (319.167 m^3/fill) / 0.5 h

= 638.33 m^3/h this is the rate at which water has to be pumped out

Now we have to determine the oxygen required, after finding this we have to estimate the rating of aerators required.

Components involved in finding out the oxygen required include,

Oxygen consumed by heterotrophs to degrade OM – Endogenous decay of MO + Oxygen required for nitrification

$$Q(S_0 - S) - 1.42P_{x,B_iO} + 4.57Q(NO_x)$$

All the while I have assumed that S_0 – S is almost equal to S_0, going to apply the same assumption here also,

Oxygen required = 875.77 kg of O_2 / d

We need to estimate the oxygen required per cycle, Total No. of cycles for one reactor per day is 6, so the oxygen required per cycle is – **145.96 kg of O_2 / Cycle**

Aeration time = 1 h

Oxygen required = (145.96 kg of O_2 / Cycle) / (1 h/cycle)

= 145.96 kg of O_2 / h

It's a well-known fact that we need not provide the same transfer rate throughout the aeration period, we could taper it also.

Estimation of the sludge produced:

Suspended solids produced per day = $\frac{\boldsymbol{V*1000(MLSS)}}{\boldsymbol{SRT}}$

= 531944333.3 mg/day (531.94 kg/day)

From the above calculation it is evident that nitrification of the effluent has been carried out within the aeration time, to carry out denitrification and phosphorus removal selector zones could be provided within the SBR, **Nitrate removal can be carried out by Cyclic aeration on/off operation during the react period**, to provide an anoxic environment for the denitrifies to get into action.

Design Summary (for 2031 flow):

Design Parameter	Unit	Value
Average Flowrate	m^3/d	3830
Number of Tanks	Number	2
Fill time	h	2
React time	h	1
Settle time	h	0.5
Decant time	h	0.5
Cycle time	h	4
SRT	days	10.5
Tank volume	m^3	1595.83
Fill volume / cycle	m^3	319.16
Decant depth	m	0.8
Tank depth	m	4
MLSS	mg/L	3500
Decant pumping rate	m^3/min	10.64
Sludge production	kg TSS/d	531.95
Average oxygen required per tank/cycle	kg/d	145.96
Average oxygen transfer rate	kg/h	145.96

Aerator Design:

$$\text{SOTR} = \left(\frac{\text{OTR}_f}{\alpha F}\right)\left[\frac{C^*_{\infty 20}}{\beta(C_{st}/C^*_{s20})(P_b/P_s)(C^*_{\infty 20}) - C}\right][(1.024)^{20-T}]$$

SOTR is the Standard Oxygen transfer rate at site

OTR is the oxygen transfer rate at site

F diffuser fouling factor

C_{st}^* - Saturated DO at sea level and operating temperature

C_{s20}^* - Saturated DO at sea level at 20°C

To estimate $C_{inf,20}^*$ US EPA has provided a formula

$$C^*_{\infty,20} = C^*_{S20}\left[1 + d_e\left(\frac{D_f}{P_a}\right)\right]$$

C_{s20}^* is 9.09 mg/L, depth of the tank is 3.5m (4-0.5) and P_a IS 10.33m, substituting the values in the above equation yields **$C_{inf,20}^*$ as 10.32 mg/L**

$$\frac{P_b}{P_a} = \exp\left[-\frac{gM(z_b - z_a)}{RT}\right]$$

Finding out the pressure at an elevation of 423m (that's the elevation of the wastewater treatment plant). M is the molecular weight of air and the temperature is 29°C.

P_b/P_a = 0.953

Next step is to estimate the Standard oxygen transfer rate, now that all parameters have been found out

OTR is found from the design of SBR, which is 145.96 kg/h, 2mg/L is the minimum DO to be maintained in the aeration basin. α = 0.5 and β = 0.95, diffuser fouling factor = 0.9

SOTR = 457.76 kg/h, subsequently calculating the air flow rate in m^3/min, with the efficiency of fine bubble membrane diffusers as 35%

Air flow rate = 84.81 m^3/min (457.76)/ (0.257*0.35*60)

Design of SBR for 2051 flow

Same influent characteristics are considered,

Finding out SBR operating cycle

Different stages in SBR include

6. Fill (t_f)
7. React (t_r)
8. Settle (t_s)
9. Decant (t_d)
10. Idle

In this step, I will be assuming the durations for each step

4. t_r = 1 h
5. t_s = 0.5 h
6. t_d = 0.5 h

Fill time would be the summation of react, settle and decant period, since I am using two SBR then the total cycle would be twice the fill time

t_f = 2 h (1+0.5+0.5)

Total cycle time = 4 h (2*t_f)

Number of cycles per day per reactor = 6 (24/4)

Total cycles combined of two reactors = 12

Fill volume per cycle = (3681 m^3/d)/(12) (difference between 2051 and 2031 flow)

= 306.75 m^3/ fill

Tank depth is assumed to be – 4m

Decant depth = 0.8 m (4*0.2)

Volume of the Tank = 1533.75 m^3 (306.75/0.2)

Hydraulic retention time = 20 h (2*1533.75 m^3 *24 h)/ (3681 m^3/day)

Actual sludge retention time, which was calculated using the formula provided in Metcalf and eddy (Pg 777) yielded 10.1 days, so **assuming the sludge retention time to be 10.5 days.**

Determining the MLVSS concentration

Using the same formula that was used for finding out SRT, can be used here with SRT as 10.5 days, which is equated to MLVSS * Volume of Tank

Taking out few important values from Table 8-14 of Metcalf and Eddy

Yield = 0.45 g of VSS/ g of bCOD

Endogenous decay coefficient = 0.12 (has to be modified for Average temperature of 29^{o}C)

= 0.17 (0.12*(1.04$^{(29-20)}$))

Yield of Nitrifiers = 0.15 g of VSS/ g of NO_X

Endogenous decay of Nitrifiers varies in aerobic and anoxic conditions, so a weighted average is preferred. During the fill time, the reactor is in aerobic condition, once the fill stage is completed nitrifiers would get into action and ammonia would be converted to nitrate, so anoxic condition is established.

Fraction of fill time with respect to the entire cycle time is – 0.25 h (1 h/ 4 h)

Endogenous decay of nitrifiers in aerobic environment – 0.17 g/gd

Endogenous decay of nitrifiers in anoxic environment – 0.07g/gd

Average Endogenous decay =<u>0.122</u> (0.17*(1.029$^{(29-20)}$) *(0.25) + 0.07*(1.029$^{(29-20)}$) *(1-0.25))

Fraction of cell mass remaining as cell debris = 0.15g/g

An expression for total solids wasted in terms of Volatile suspended solids is provided in Metcalf and Eddy denoted as $P_{x,VSS}$ (g of VSS/d)

Now that we have the value for SRT, ($P_{x,VSS}$ * SRT) would yield Total MLVSS (g) in the aeration tank

Equation for $P_{x,VSS}$

Would be contributed by Heterotrophic biomass, Cell debris, Non-biodegradable VSS in effluent and Nitrifiers

$$= \frac{Qy_H(s_0 - s)SRT}{1 + b_H(SRT)} + Q * nbVSS(SRT) + \frac{QY_n * NO_X(SRT)}{1 + b_n(SRT)} + \frac{f_d b_H QY_H * (S_0 - S)SRT^2}{1 + b_H(SRT)}$$

Substituting the necessary data into the equation (SRT taken as 10.5 days) yields

$P_{x,VSS}$ = 3904129.95 g

MLVSS (g/m^3) = 2545.5 g/m^3 (mg/L) (3904129.95/1533.75)

MLVSS/MLSS = 0.727 (2545.5 mg/L /3500 mg/L)

Next step would be to identify the nitrogen that is oxidized which is given as

Nitrogen in the influent – Nitrogen in the effluent – Nitrogen in cell mass

First step would be to identify the total cell mass generated per day which is given by $P_{x,bio}$

$$\frac{Qy_H(s_0-s)}{1+b_H(SRT)} + \frac{QY_n*NO_X}{1+b_n(SRT)} + \frac{f_d b_H QY_H*(S_0-S)SRT}{1+b_H(SRT)}$$

Cell mass produced per day = 196406.1 g/day

= 196.406 kg/day

Inflow TKN is 35 mg/L

Effluent nitrogen required is 1 mg/L

Amount of Nitrogen oxidized = 21.194 mg/L (35-1-((0.12*196406.1)/ (1840.5)))

The task would be to find if the Level of nitrification reach 1 mg/L in the given span of 1h for aeration

Oxidizable NH_4-N added per cycle = (306.75*21.19*1000)

=6501376.66 mg/fill

NH_4-N remaining in the tank before the fill = 1*(1533.75-306.75) *1000

= 1227000 mg/fill

Total oxidizable N at the Beginning of the cycle = (6501376.66+1227000) mg/fill

= 7728376.66 mg/fill

Initial concentration = 5.038 mg/L (7728376.66/ (1533.75*1000))

We need to know the quantity of nitrifiers in the system that would perform the oxidation of ammonium nitrogen to nitrate and nitrite nitrogen

$$\frac{QY_n * NO_X(SRT)}{1 + b_n(SRT)}$$

Nitrifier concentration = 17.49 mg/L

Kinetic co-efficient for nitrifiers were extracted from Table 8-14 of Metcalf and Eddy

$\mu_{m,\,29}$ =1.682 d^{-1} (0.9*$(1.072)^{29-20}$), $K_{NH4\,29}$ = 0.934 g/m^3 (0.5*$(1.072)^{(29-20)}$), K_o = 0.5 g/m^3

$$k_{NH_4} \cdot \ln\left(\frac{N_0}{Nt}\right) + (N_0 - N_t) = x_n\left(\frac{\mu_{m_{ax}}}{y_n}\right)\left(\frac{s_0}{k_0 + s_0}\right)t$$

N_0 = 4.395 mg/L, N_t = 1 mg/L, S_0 is the minimum dissolved oxygen in the wastewater

S_0 = 2 mg/L

Finding t from the equation is the objective and we have to check if that value is less than the aeration time, if it's less then we have to proceed with finding the oxygen requirement and aeration design.

After substituting the values, t from the above equation is 0.848 h which is less than the aeration time of 1 h hence there is sufficient time to oxidize the incoming nitrogen to the prescribed standards.

Decant pumping rate = Fill volume per cycle / decant time

= (306.75 m^3/fill) / 0.5 h

= 613.5 m^3/h this is the rate at which water has to be pumped out

Now we have to determine the oxygen required, after finding this we have to estimate the rating of aerators required.

Components involved in finding out the oxygen required include,

Oxygen consumed by heterotrophs to degrade OM – Endogenous decay of MO + Oxygen required for nitrification

$$Q(S_0 - S) - 1.42P_{x,B_iO} + 4.57Q(NO_x)$$

All the while I have assumed that $S_0 - S$ is almost equal to S_0, going to apply the same assumption here also,

Oxygen required = 841.706 kg of O_2 / d

We need to estimate the oxygen required per cycle, Total No. of cycles for one reactor per day is 6, so the oxygen required per cycle is – **140.284 kg of O_2 / Cycle**

Aeration time = 1 h

Oxygen required = (140.384 kg of O_2 / Cycle) / (1 h/cycle)

= 140.384 kg of O_2 / h

It's a well-known fact that we need not provide the same transfer rate throughout the aeration period, we could taper it also.

Estimation of the sludge produced:

Suspended solids produced per day = $\frac{\boldsymbol{V*1000(MLSS)}}{\boldsymbol{SRT}}$

= 511250000 mg/day

= 511.25 kg/day

From the above calculation it is evident that nitrification of the effluent has been carried out within the aeration time, to carry out denitrification and phosphorus removal **selector zones**

could be provided within the SBR, Nitrate removal can be carried out by Cyclic aeration on/off operation during the react period, to provide an anoxic environment for the denitrifies to get into action.

Design summary of SBR for 2051 Flow

Design Parameter	Unit	Value
Average Flowrate	m^3/d	3681
Number of Tanks	Number	2
Fill time	h	2
React time	h	1
Settle time	h	0.5
Decant time	h	0.5
Cycle time	h	4
SRT	days	10.5
Tank volume	m^3	1533.75
Fill volume / cycle	m^3	306.75
Decant depth	m	0.8
Tank depth	m	4
MLSS	mg/L	3500
Decant pumping rate	m^3/min	10.22
Sludge production	kg TSS/d	511.25
Average oxygen required per tank/cycle	kg/d	140.38
Average oxygen transfer rate	kg/h	140.38

Aerator Design:

$$\mathrm{SOTR} = \left(\frac{\mathrm{OTR}_f}{\alpha F}\right)\left[\frac{C^*_{\infty 20}}{\beta(C_{st}/C^*_{s20})(P_b/P_s)(C^*_{\infty 20}) - C}\right]\left[(1.024)^{20-T}\right]$$

SOTR is the Standard Oxygen transfer rate at site

OTR is the oxygen transfer rate at site

F diffuser fouling factor

C_{st}^* - Saturated DO at sea level and operating temperature

C_{s20}^* - Saturated DO at sea level at 20°C

To estimate $C_{inf,20}^*$ US EPA has provided a formula

$$C^*_{\infty,20} = C^*_{S20}\left[1 + d_e\left(\frac{D_f}{P_a}\right)\right]$$

C_{s20}^* is 9.09 mg/L, depth of the tank is 3.5m (4-0.5) and P_a IS 10.33m, substituting the values in the above equation yields **$C_{inf,20}^*$ as 10.32 mg/L**

$$\frac{P_b}{P_a} = \exp\left[-\frac{gM(z_b - z_a)}{RT}\right]$$

Finding out the pressure at an elevation of 423m (that's the elevation of the wastewater treatment plant). M is the molecular weight of air and the temperature is 29°C.

P_b/P_a = 0.953

Next step is to estimate the Standard oxygen transfer rate, now that all parameters have been found out

OTR is found from the design of SBR, which is 140.38 kg/h, 2mg/L is the minimum DO to be maintained in the aeration basin. α = 0.5 and β = 0.95, diffuser fouling factor = 0.9

SOTR = 440.26 kg/h, subsequently calculating the air flow rate in m^3/min, with the efficiency of fine bubble membrane diffusers as 35%

Air flow rate = 81.57 m^3/min (440.26)/ (0.257*0.35*60)

Tertiary Treatment

Multi Grade Filters:

From the field visit to Nesapakkam STP plant, I understood the various tertiary systems in play, I would like to emulate similar technology in my design also, I would be using Multi grade filters, that have varying particle sizes across the depth of the filter. Anthracite at the top followed by fine sand, fine gravel and large gravel. It is used to remove suspended solids and turbidity from the SBR effluent.

When multi grade filters are used, we need to estimate the head loss occurring as water flows through it and the backwash velocity and the depth of the expanded bed. Particles in the water are filtered out at various depths in a multimedia filter, the filter does not clog as quickly as if all of the particles were all caught by the top layer. Effluent from MGF could be used for gardening, toilet flushing and cleaning purposes.

Multi grade filters have higher specific flow rates than conventional filters, specific flow rates of 0.82 – 1.64 ft/min have been successfully obtained.

Rate of Filtration = 20 $m^3/m^2/h$ (anything between 15-30 $m^3/m^2/h$)

Assuming that filter is backwashed 0.5 h per day, so the total operating time would be 23.5h

Total filtered water within this time period would be =162.978 m^3/h ((3.83*1000 m^3/d)/(23.5h/d))

Area required =8.15 m^2 ((162.978 m^3/h)/ (20 $m^3/m^2/h$))

So, providing one filter with a **diameter of 3.25 m**

Next step is to estimate the head loss, assuming a gradation table given below

Sieve No		Particle size range (mm)		Average Size (dij, mm)	Mass fraction in size range (xij)
Passing	Retained	Passing	Retained		
-	14	-	1.41	1.41	0.01
14	20	1.41	0.84	1.13	0.11
20	25	0.84	0.71	0.78	0.20
25	30	0.71	0.60	0.66	0.32
30	35	0.60	0.50	0.55	0.21
35	40	0.50	0.42	0.46	0.13
40	-	0.42	-	0.42	0.02

$$\mathrm{Re} = \frac{\phi e_{\omega} v_s d}{\mu}$$

Dynamic viscosity of water at 29°C is $0.8145*10^{-3}$ kg/m.s

In the above Reynold's equation is a function of dia

Shape factor is taken as 0.85, substituting other parameters in Re, final value would be

$$\mathbf{Re = 5787.25\ d_{ij}}$$

$$f' = \frac{150(1-e)}{\mathrm{Re}} + 1.75$$

Using the friction factor equation, which can be written as a function of dij, assuming the porosity as 0.4

$$\frac{0.0155}{d_{ij}} + 1.75$$

Head loss in the filter for multimedia is given by the equation

$$\mathrm{H_f} = \frac{L(1-e)v_s^2}{e^3 g} \Sigma \frac{f_{ij} x_{ij}}{d_{ij}}$$

$$\Sigma f_{ij}\left(\frac{x_{ij}}{di_{ij}}\right)$$

Summation of the above term = 42661.59

dij	xij	fij	fij(xij/dij)
1.41	0.01	12.74291	90.37523
1.13	0.11	15.46681	1505.619
0.78	0.2	21.62179	5544.05
0.66	0.32	25.23485	12235.08
0.55	0.21	29.93182	11428.51
0.46	0.13	35.44565	10017.25
0.42	0.02	38.65476	1840.703
			42661.59

Assuming that the depth of the filter is 0.75m

Head loss in the filter = 0.95m

Next step would be to estimate the depth of the expanded bed, we need to estimate the velocity in each layer, for this we have to initially assume it is turbulent and check for Reynold's number, if it doesn't satisfy, we have to proceed with transitional and iterate until the point where velocity difference between subsequent iterations is less than a certain value. Assuming that d_{ij} of 0.46 and 0.42mm belong to anthracite with a specific gravity of 1.5, which is placed at the top and sand of varying size from 0.55 to 1.41 mm placed beneath it with a specific gravity of 2.65. All of these assumptions were taken into consideration while finding out the velocity in each layer. Final velocity after many iterations has been tabulated below.

dij	Vij	1-((Vb/Vij)^0.22)	xij	xij/(1-((Vb/Vij)^0.22)
1.41	0.23	0.436208351	0.01	0.022924825
1.13	0.22	0.430667766	0.11	0.255417305
0.78	0.206	0.42237235	0.2	0.473515845
0.66	0.198	0.417316902	0.32	0.766803354
0.55	0.189	0.411322867	0.21	0.510547836
0.46	0.079	0.286784451	0.13	0.453302121
0.42	0.076	0.280683914	0.02	0.071254529
				2.553765814

Length of the expanded bed = 1.14m (0.75(1-0.4) *(2.553))

Design Summary:

1 Multi Grade filter with anthracite of size 0.42 and 0.46mm with a specific gravity of 1.5 is placed at the top followed by sand of size from 0.55 to 1.41mm with a specific gravity of 2.65 is placed beneath anthracite.

Rate of filtration = 20 $m^3/m^2/h$

Depth of the filter = 0.75m

Diameter of the unit = 3.25m

Backwash velocity = 0.017m/s

Expanded bed depth = 1.14m

Once this is established for 2041 flow and 2051 flow only the number of Multi grade filters would change, whereas the depth, backwash velocity, gradation and velocity distribution would remain the same.

Effluent from Multi grade filter could be transported to clients, who require water for flushing purposes.

For 2051 flow:

Rate of Filtration = 20 $m^3/m^2/h$ (anything between 15-30 $m^3/m^2/h$)

Assuming that filter is backwashed 0.5 h per day, so the total operating time would be 23.5h

Total filtered water within this time period would be =156.63 m^3/h ((3.681*1000 m^3/d)/(23.5h/d))

Area required =7.83 m^2 ((332.127 m^3/h)/ (20 m^3/m^2/h))

So, providing one filter with a **diameter of 3.25 m**

Design summary:

1 Multi Grade filter (already one is installed) with anthracite of size 0.42 and 0.46mm with a specific gravity of 1.5 is placed at the top followed by sand of size from 0.55 to 1.41mm with a specific gravity of 2.65 is placed beneath anthracite.

Rate of filtration = 20 m^3/m^2/h

Depth of the filter = 0.75m

Diameter of the unit = 3.25m

Backwash velocity = 0.017m/s

Expanded bed depth = 1.14m

Ultrafiltration

TDS in the influent wastewater is 1550mg/l, let's assume the effluent TDS to be equal 100mg/l.

Parameters assumed:

1. Solvent mass transfer coefficient = $9*10^{-9}$ s/m (K_w)
2. Solute mass transfer coefficient = $6*10^{-8}$ m/s (K_i)
3. Effluent TDS = 100mg/l
4. Influent TDS = 1500mg/l
5. Feed water flow rate = 0.0443 m^3/s
6. Net operating pressure = 2500 kPa ($2.5*10^6$ kg/ms^2)
7. Recovery rate = 90%

2031 Flow:

Area of the membrane =1772 m^2 (0.9*0.0443 m^3/s * 1000 kg/m^3)/ ($9*10^{-9}$ s/m * $2.5*10^6$ kg/m.s^2)

Area of one membrane = 81m^2

So, providing 22 UF Membranes with element diameter of 280mm

Finding out the permeate TDS for the given area of Membrane

$$C_P = \frac{k_i\left(\left(C_f + Cr\right) / 2\right)A}{Qp + k_iA}$$

Concentration of TDS in the feed is known and assuming that Concentration in the retentate as 10 times the concentration of the feed (this is an iterative procedure).

C_p (Concentration in the permeate) = 19.752 mg/L

Rejection Rate = 98.68%

Estimating the concentration of the retentate

$$C_rQ_r + C_pQ_p = C_fQ_f$$

Concentration in the retentate = 14822.23 mg/L, whereas assumed concentration of retentate was 15000 mg/L, which is pretty close, when we take other multiples of C_f as concentration of retentate, it does not match

Multiples	Concentration of permeate	Calculated Concentration of retentate	Actual Concentration of retentate
1	3.591380686	14967.67757	1500
2	5.38707103	14951.51636	3000
3	7.182761373	14935.35515	4500
4	8.978451716	14919.19393	6000
5	10.77414206	14903.03272	7500
6	12.5698324	14886.87151	9000
7	14.36552275	14870.7103	10500
8	16.16121309	14854.54908	12000
9	17.95690343	14838.38787	13500
10	19.75259377	14822.22666	15000
11	21.54828412	14806.06544	16500
12	23.34397446	14789.90423	18000
13	25.1396648	14773.74302	19500
14	26.93535515	14757.5818	21000
15	28.73104549	14741.42059	22500

Concentration of retentate as 10 times the concentration of feed is perfect.

Design Summary:

Provide 22 UF membranes, effluent TDS is 19.75 mg/L with a recovery rate of 90% and rejection rate of 98.68%. Retentate is taken back to the equalization tank

2051 Flow:

Area of the membrane =1704 m^2 (0.9*0.0426 m^3/s * 1000 kg/m^3)/ (9*10^{-9} s/m * 2.5*10^6 $kg/m.s^2$)

Area of one membrane = 81m^2

So, providing 22 UF Membranes with element diameter of 280mm

Finding out the permeate TDS for the given area of Membrane

$$C_P = \frac{k_i\left(\left(C_f + Cr\right) / 2\right)A}{Qp + k_iA}$$

Concentration of TDS in the feed is known and assuming that Concentration in the retentate as 10 times the concentration of the feed (this is an iterative procedure).

C_p (Concentration in the permeate) = 19.752 mg/L

Rejection Rate = 98.68%

Estimating the concentration of the retentate

$$C_rQ_r + C_pQ_p = C_fQ_f$$

Concentration in the retentate = 14822.23 mg/L, whereas assumed concentration of retentate was 15000 mg/L, which is pretty close, when we take other multiples of C_f as concentration of retentate, it does not match.

Effluent from UF could be taken to industries for low end uses like cooling, it can be sold to construction sites also. Adding another module of Reverse osmosis could produce Grade III water, which is used as process water in many industries.

Design Summary:

Provide 22 UF membranes (already 22 are UF membranes are in place and working), effluent TDS is 19.75 mg/L with a recovery rate of 90% and rejection rate of 98.68%. Retentate is taken back to the equalization tank

Biosolids Management

Sludge is an integral part of the management plan of any water treatment plant, as they have the potential to impact the public health. In my design sludge would be collected from SBR only, as I have not provided PST and SST. Materials collected in the grit chamber would be subjected to segregation and proper disposal pathway would be adopted.

In 2031 flow, amount of sludge generated per day from SBR is 531.95 kg/d, in 2051 flow it's around 511.25 kg/d. Sludge in general contains predominantly water and less percentage of solids. It is necessary to subject the sludge to various pre-treatment processes before putting into anaerobic digesters.

Processes involved

1. **Grinding of sludge**
   Large and stingy material in sludge is sheared off to smaller particles to prevent wrapping or clogging around rotating equipment.
2. **Thickening of sludge**
   It's a process to increase the solid content in the sludge by removing the liquid portion. It's mostly accomplished by physical means like centrifuging, gravity settling and gravity belt, volume reduction taking place would benefit the subsequent processes like digestion, dewatering and drying beds. **In this design I would be adopting centrifuging for thickening of sludge. Final solids content after thickening would be around 2-3%**
3. **Sludge stabilization** – In this design I would be adopting **anaerobic digestion**.
4. Biosolids from the anaerobic digestion is subjected to **chemical conditioning (**process of adding chemicals to the sludge so that coagulation of solids take place and bound water is released**) and Filter press** is used to squeeze out the water, filtrate could be taken to the equalization tank and the final sludge material is given to State Forest service college and Tamil Nadu agricultural university or it is taken to Landfill in Vellalore.

Anaerobic digestion:

Single stage, High-rate digester is used for the sludge stabilization

For 2031 flow:

Wastewater flow handled = 3830 m^3/d

Influent BOD = 320 mg/L

Influent TSS = 350 mg/L

Specific gravity of the sludge = 1.02

SRT is decided based upon the ambient temperature for 29-30°C Desired SRT is 14 days, so providing **15 days as the SRT**

Assuming that sludge contains sufficient nitrogen and phosphorus for biological growth (this is true as nitrification has taken place in SBR and phosphorus is untouched)

Thickened sludge has **solid content of 3%**

Endogenous coefficient = 0.03 d^{-1}

Sludge mass produced daily = 531.95 kg/d

Sludge volume = 17.383 m^3/d (531.95/ (1.02*0.03*1000))

bCOD loading = 1960.96 kg/d ((1.6*320*3830*1000)/ (1*10^6))

Digester Volume = 260.75 m^3 (17.383 m^3/d*15d)

Volumetric loading = 7.52 kg/m^3. d (1960.96 kg/d / 260.75 m^3)

Quantity of volatile solids produced per day = 108.19 kg/d

Volume of methane produced per day, Conversion **factor for 29°C is 0.387**

Volume of methane produced = 699.4 m^3/d

Assuming that digester gas is 60% methane, total volume of the digester gas = 1165.6 m^3/d

Design Summary:

Providing 1 digestor with a diameter of 8.5m and depth of 5m, with methane production of 699.4 m^3/d

For 2051 flow:

Wastewater flow handled = 3681 m^3/d

Influent BOD = 320 mg/L

Influent TSS = 350 mg/L

Specific gravity of the sludge = 1.02

SRT is decided based upon the ambient temperature for 29-30°C Desired SRT is 14 days, so providing **15 days as the SRT**

Assuming that sludge contains sufficient nitrogen and phosphorus for biological growth (this is true as nitrification has taken place in SBR and phosphorus is untouched)

Thickened sludge has **solid content of 3%**

Endogenous coefficient = 0.03 d^{-1}

Sludge mass produced daily = 511.25 kg/d

Sludge volume = 16.70 m^3/d (511.25/ (1.02*0.03*1000))

bCOD loading = 1884.67 kg/d ((1.6*320*3681*1000)/ (1*10^6))

Digester Volume = 250.5 m^3 (16.70 m^3/d*15d)

Volumetric loading = 7.523 kg/m^3. d (1884.67 kg/d / 250.5 m^3)

Quantity of volatile solids produced per day = 103.98 kg/d

Volume of methane produced per day, Conversion **factor for 29^{o}C is 0.387**

Volume of methane produced = 694.80 m^3/d

Assuming that digester gas is 60% methane, total volume of the digester gas = 1158 m^3/d

Design Summary:

Providing 1 digestor with a diameter of 8.5m and depth of 5m, with methane production of 694.8 m^3/d

1 m^3 biogas at 60% methane content has 22MJ energy

At 35% electrical energy conversion, it will yield 2.14 kWh, roughly we could say that,

1.284 kWh/ cubic metre biogas per day

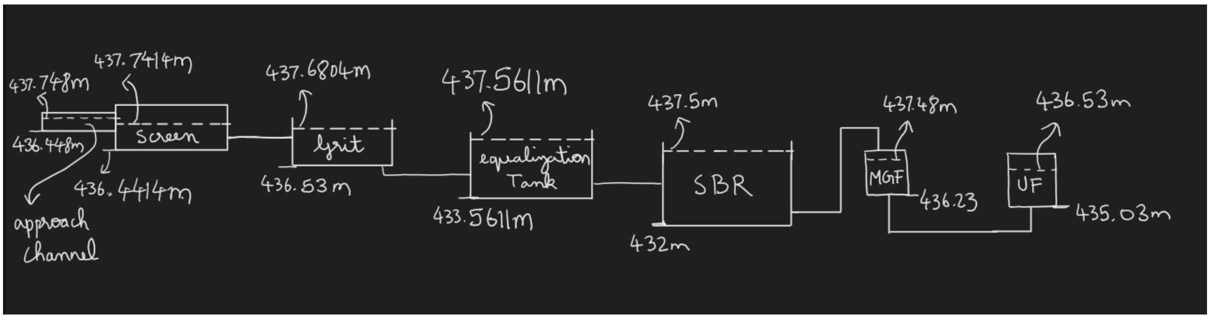
From these calculations we can infer that up to 2031 we could produce electrical energy of 1496.63kWh could be produced per day.

For 2051 total biogas produced would be 2323.6 m^3/d we could produce electrical energy of 2983.50kWh per day

Hydraulic Profiling

	L	S	Hf	Hm	Hf+Hm
Approach channel	3.5	0.002047926	0.007167741		0.007168
Head loss when ww passes through screen			0.0106		0.0106
Head loss from screen to grit chamber (Pipe)	5	0.003205398	0.016026991		0.016027
Entry Head loss grit chamber				0.034469139	0.034469
Exit head loss grit chamber				0.068938277	0.068938
Grit to equalization pipe head loss	5	0.003205398	0.016026991		0.016027
Entry head loss equalization tank				0.034469139	0.034469
Exit head loss equalization tank				0.033605708	0.033606
Equalization tank to SBR pipe head loss	5	0.002148583	0.010742915		0.010743
SBR Entry head loss				0.016802854	0.016803
SBR Exit head loss				0.033605708	0.033606
Head loss in MGF				0.95	0.95
Head loss in pipe MGF TO UF	5	0.002148583	0.010742915		0.010743
				1.171890826	1.243198

Let's assume the final reactor SBR to be at an elevation of



Calculations Involved in hydraulic profiling

1. Head loss when WW travels through the pipe, it's found out using **chezy's equation** by **estimating S_f**, this multiplied with the length of the pipe would provide the Head loss as it travels through the pipe. I am also assuming that **d/D ratio is equal to 0.8**, sewers when it's flowing full might induce anaerobic condition which would favour SRB for the generation of H_2S gas, which would lead to **crown corrosion**.
2. When WW enter and exits the reactor, there would be head loss in these instances, **velocity in the pipe is known**, that would be used to calculate **entry and exit head loss**.
3. Head loss occurring as the Effluent flows through the MGF is calculated as 0.95m and the same is used here for finding out the water level in UF
4. SBR is placed at ground level and back calculation is carried out to the find the elevation of water level and reactor base level in the upstream.
5. Usually, the initial reactors would be placed at a higher level to enable the gravity flow, to make sure cost is diverted from pumping WW. This was observed in IITM STP plant, WW was pumped to a higher level to the grit chamber and then flow was governed by gravity.

Few Calculations:

Water level in Equalization Tank: 437.5 (Water level in SBR) + 0.0107 (Head loss as it's transported via pipe) + 0.0168 (SBR Entry head loss) + 0.0336 (Equalization Tank exit head loss) = 437.5611m

Water level in Grit chamber: 437.5947m (Water level in equalization tank) + 0.016m (Head loss as WW is transported from grit to equalization tank) + 0.0344m (Entry head loss in equalization tank) + 0.068m(exit head loss in Grit Chamber) = 437.6804m

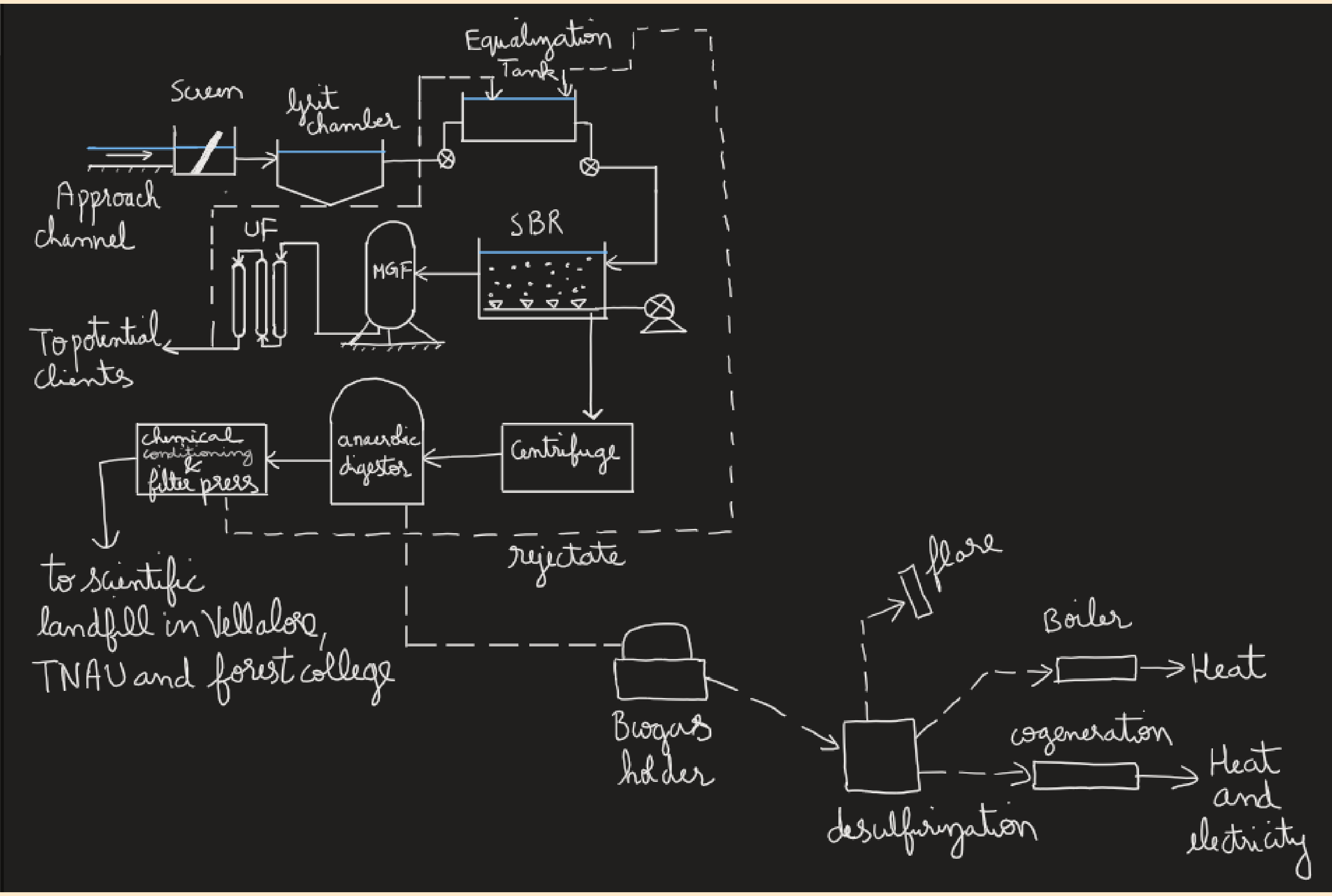
Water level in Screen: 437.7826m (Water level in Grit chamber) +0.0344m (Entry head loss in grit chamber) + 0.0106m (Head loss in screens) +0.016m (Head loss when WW is transported via the pipe) = 437.7414m

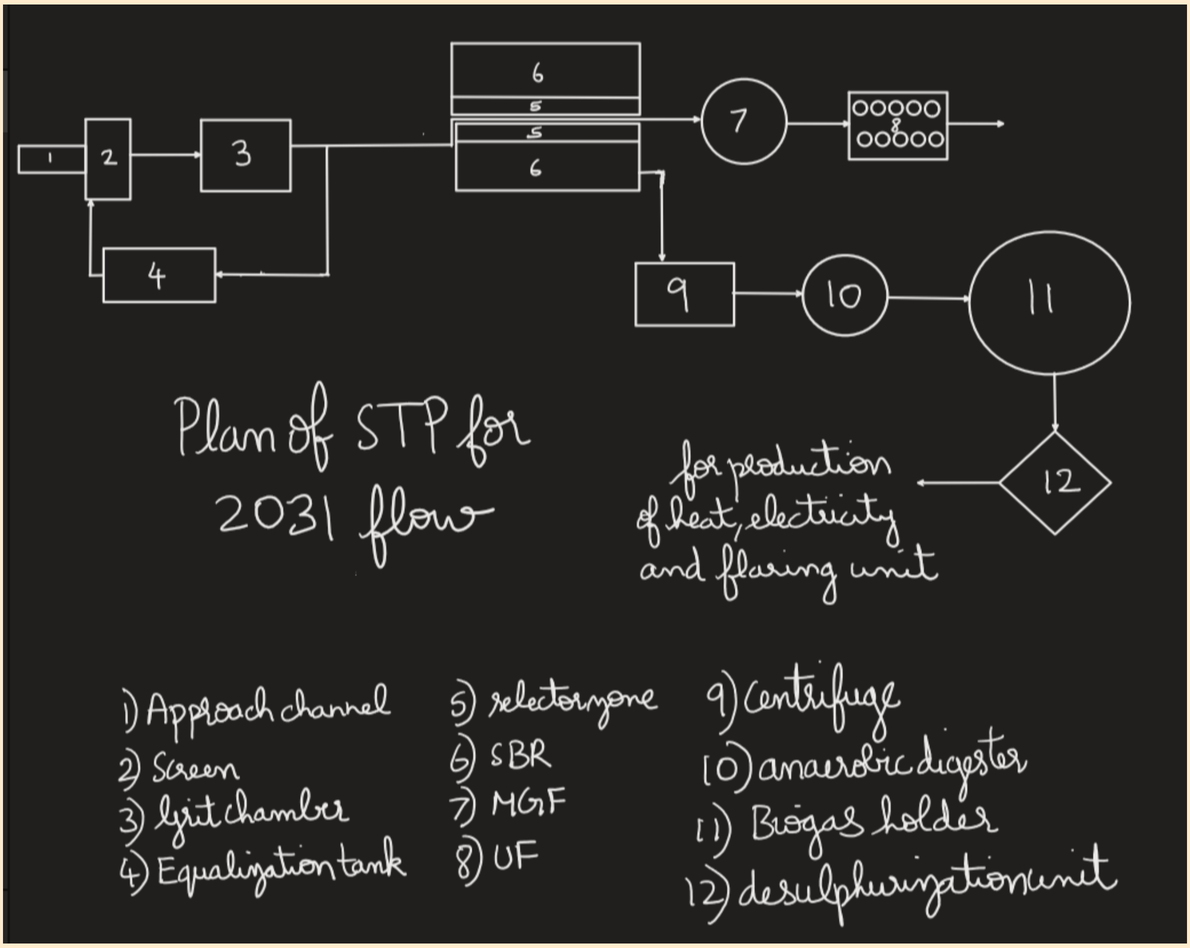
Diameter of the pipes transporting WW from screen to grit and grit to equalization tank have a diameter of 0.5m with depth of flow maintained at a d/D ratio of 0.8 so the depth of flow is 0.4m.

Diameter of the pipes transporting WW from equalization to SBR and SBR to MGF and subsequent reactors have diameter of 0.4m with the depth of flow being 0.32m

It is evident from the hydraulic profiling, the initial reactors have to be placed at a higher level, so that the rest of the flow happens via gravity and the cost of pumping is reduced.

UNITS IN SEWAGE TREATMENT PLANT AND SITE MAP





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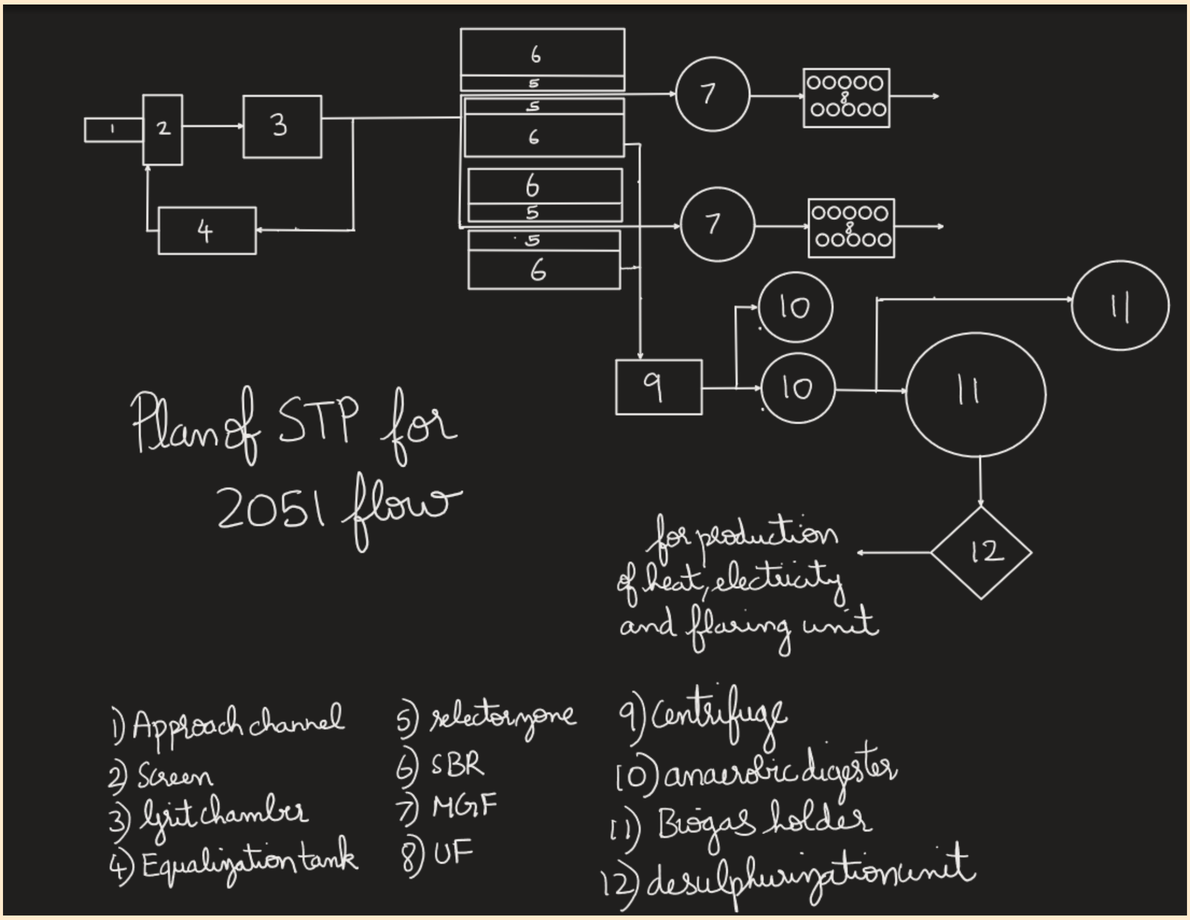
Plan of STP for
2031 flow
for production
of heat, electricity
and flaring unit
1) Approach channel
2) Screen
3) Grit chamber
4) Equalization tank
5) selector zone
6) SBR
7) MGF
8) UF
9) Centrifuge
10) anaerobic digester
11) Biogas holder
12) desulphurization unit



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Plan of STP for
2051 flow
for production
of heat, electricity
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1) Approach channel
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5) selector zone
6) SBR
7) MGF
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