WATER AND SANITATION AGENCY (WASA), GUJRANWALA

FEASIBILITY REPORT

Feasibility Study, Planning and Design of Wastewater Treatment Plants in Gujranwala City

April, 2013

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Executive Summary

Presently, no sewage treatment facility exists in Gujranwalaand the raw wastewateris being disposed, without any treatment, intoirrigation channels/ drains. It is causing environmental hazards formankind and adversely affecting the water quality of receiving waters. Keeping in view the above situation, Water and Sanitation Agency (WASA), Gujranwala, engaged Jers Engineering Consultants to provide consultancy services regarding the feasibility study, planning and design of wastewater treatment plant/s (WWTP) for the Gujranwala up to the design year 2038.

Gujranwala city is provided with sewerage system. Presently there are 16 sewage disposal stations, which receive wastewater from collection network and dispose it into five main drains (Main drain, QilaMian Singh, Jinnah drain, Mir Shikara and AduRai drain). These drains finally dispose the wastewater into Qadirabad Baloki Link Canal.

Different treatment technologies are available for the treatment of wastewater. As per the requirement of TOR, six different treatment technologies are evaluated in this feasibility report viz. Activated sludge process, Upflow Anaerobic Sludge Blanket, Biological Trickling Filter, Oxidation Ditch, Aerated Lagoon and Waste Stabilization Ponds. Based on the comparison, Waste stabilization ponds (WSP) are recommended for Gujranwala. It is cheaper option with respect to capital and operation cost. Additional benefit of WSP is that it does not have energy requirement.

With respect to wastewater/sewage treatment two possibilities exist viz. (1) a combined WWTP for the entire city and (2) multiple WWTPs treating wastewater for each main drain. The possibility of having multiple WWTPs seems more feasible due to two reasons viz. (1) This would help to break down the entire cost and take up prioritized works one by one depending upon the availability of funds; (2) it would also be difficult to acquire one large piece of land at one place. Due to congestion and non-availability of land inside the city, the WWTP/s have to be shifted outside. Moreover, shifting of WWTP/s outside city would be environmentally desirable.

Based upon the above concept both possibilities have been worked out i.e. combined and multiple WWTP/s. The summary of area requirement and cost for WSP has been shown in the Table below.

Name of WWTP	Total Area of WWTP		Capital cost (Rs Million)	O&M cost (Rs Million)
	Hectare	Acre		
Option-1 (combined WWTP)				
Combined WWTP for whole city	430	1062	5913	35.8
Option-2 (Multiple WWTPs)				
Qila Mian Singh	174	430	2187	12.6
Main Drain & Jinnah Drain*	260	641	3267	19.5
Mir Shikara	114	282	1157	4.6
AduRai	17	41	195	1.0

**Separate treatment of Main and Jinnah drain is not possible*

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List of Abbreviations

ASP	Activated Sludge Process
AE	Aerated Lagoon
AP	Anaerobic Pond
m^3	Cubic Meter
°C	Degree Centigrade
DO	Dissolved Oxygen
XEN	Executive Engineer
FP	Facultative Pond
BOD	Five day Biochemical Oxygen Demand
JEC	Jers Engineering Consultant
MP	Maturation Pond
m	Meter
mg/L	Milligram/Liter
OD	Oxidation Ditch
PHED	Public Health Engineering Department
SDO	Sub Divisional Officer
SE	Superintending Engineer
BTF	Biological Trickling Filter
USA	United States of America
V:H	Vertical:Horizontal
WASA	Water and sanitation agency
WSP	Wastewater stabilization ponds
WWTP	Wastewater treatment plant

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Chapter- 1

INTRODUCTION

1.1 THE PROJECT

Presently, no sewage treatment facility exists in Gujranwalaand the raw wastewateris being disposed, without any treatment, in to irrigation channels/ drains which is causing environmental hazards for mankind. Provision of wastewater treatment facility is an obligation as per Pakistan Environmental Protection Laws and regulation for discharge of effluents. The wastewater to be disposed into recipient bodies is required to be treated as per National Environmental Quality Standards (NEQS).

Keeping in view the above situation, Water and Sanitation Agency (WASA), Gujranwala, in association with Planning and Development (P&D) Department, Punjab engaged Jers Engineering Consultants to provide consultancy services regarding the feasibility study, planning and design of wastewater treatment plant/s (WWTP)for the Gujranwala up to the design year 2038.

1.2 PROJECT OBJECTIVES

The objective of the project is to prepare a comprehensive long term plan/feasibility to provide a cost effective solution for the wastewater treatment of the entire city of Gujranwala. Therefore, as mentioned above, the project horizon is year 2038. However, construction of treatment facility for the entire city at one time is not feasible. It is due to involvement of huge amount of funds. Therefore, the feasibility will be prepared to propose short term, medium term and long term solutions. Detailed design will be carried for the complete project horizon, however, cost estimates and PC-1 will be prepared for short term interventions (high priority areas; page 5 of TOR)) as per TOR. The medium and long term steps will be initiated as and when funds will be available.

Chapter- 2

PROJECT AREA AND SCOPE OF WORK

2.1 LOCATION

Gujranwala is a historical and cultural city situated in the north east region of Punjab province. It is 63 Km from Lahore and 200 Km from Rawalpindi/Islamabad. Location Plan of the city is attached as Figure 2.1.

Gujranwala is the fifth largest city of Pakistan with a present population of about 1,662,000 . It is located at 32.16ºNorth, 74.18º East and is 226 meters (744 feet) above sea level. Gujranwala city is the divisional headquarter of the Gujranwala, Hafizabad, Sialkot, Gujrat and Narowal Districts. It is situated on the main railway line connecting Lahore and Peshawar. There are three railway stations namely Gujranwala city, Gujranwala and Gujranwala Cantt.

The Grand Trunk Road runs parallel to the railway line and passes through the centre of the city; mostly the old city is being on the west and new abadies on the east.

2.2 CLIMATE

The climate of the city is hot and dry during summer and moderately cold in winter. The summer season starts in April and continues till September. June is the hottest month with mean maximum and minimum temperatures of 40 and 27 degree centigrade, respectively. The winter season begins in November and lasts till March. January is the coldest month. The mean maximum and minimum temperatures during this month are 19 and 5 centigrade respectively. The sky is frequently overcast during winter with meager rainfall. The monsoons set in July and continue till September. The eastern part of the district receives more rain. The average annual rainfall in the district during 1961-98 is about 628.7 millimeters[1].

[1] Census report (1998), page 3.

Fig. 2.1: Location plan of Gujranwala city

2.3 PROJECT AREA

The project area of the Gujranwala city (WASA Gujranwala Jurisdiction)is about 15,000 acre. Gujranwala city is divided into four towns namely; Town-I, Town-II, Town-III and Town-IV as shown in Figure 2.2 namely;Aroop Town, Nandipur Town, KhayaliShahpur Town and QilaDidar Singh Town.

2.4 SCOPE OF WORK

The scope of work of consultants as per Term of Reference (TOR) is given below:

(i) Collection and review of available studies on wastewater treatment.
(ii) Survey and base map preparation.
(iii) Estimation and projection of population and wastewater flows
(iv) Sewage characterization.
(v) Examine various treatment options along with their technical/financial analysis for the treatment of wastewater. Help in selecting the most feasible option.
(vi) Detailed feasibility study for construction of wastewater treatment plant/s (WWTP) in the light of existing sewerage network and wastewater generation
(vii) Identification of suitable location for WWTP/s
(viii) Land assessment for WWTP/s
(ix) Geo-technical investigations at selected sites after these sites are acquired by WASA.
(x) Preparation of preliminary and detailed designing for project horizon.
(xi) BOQ, cost estimate, specification, tender documents and PC-1 for high priority WWTP/s.
(xii) Standard operating procedure (SOP) for operation and maintenance.
(xiii) IEE/EIA report.

2.5 DELIVERABLES

Following are the main deliverables of the project

1. Inception Report
2. Feasibility study report
3. Preliminary engineering design report
4. Detailed engineering design report
5. Technical specifications, cost estimate and BOQ and PC-1
6. O&M and QA/QC manual
7. Environmental report

Chapter- 3

SEWERAGE DISPOSAL POINTS AND MAIN DRAINS IN GUJRANWALA

3.1 DISPOSAL STATIONS IN GUJRANWAL

Wastewater collection system in the form of sewer pipes and sewage pumping stations exist in Gujranwala city. The wastewater is collected and then lifted with sewage pumping stations (SPS) and disposed into five main drains. A list of disposal station is given in the Table 3.1. The table also shows the drain in which the wastewater is discharged. Present pumping capacity of each SPS is shown. Present average flow has been calculated by dividing the present pumping capacity with peak factor, which is assumed as 4. A location plan showing the SPS is shown in Fig. 3.1.

Table 3.1: List of existing sewage disposal stations in Gujranwala city

Sr.No.	Disposal Station	Disposal Station pumping capacity (Cusecs)	Present Average Flow (2013) (Cusecs)	Disposal in Drain
1	Khayali	70	17.5	Main Drain
2	Sarfaraz Colony	10	2.5	
3	Peoples Colony	80	20	
4	Ittefaq Colony	5	1.25	
5	PMU	50	12.5	
6	Ferozwala	5	1.25	
7	Abu Bakar Park	5	1.25	
	Sub-Total-1	**225**	**56.25**	
8	NowsheraSansi Road	40	10	Jinnah Road Drain
9	Mughalpura	8	2	
	Sub-Total-2	**48**	**12**	
10	Samanabad	10	2.5	QilaMian Singh Minor & Mir Shikaran Drain
11	Rajkot	120	30	
12	Garjakh	28	7	
13	AlamChowk 1	8	2	
14	AlamChowk 2	30	7.5	
15	Nowshera Road	40	10	
16	Zahid Colony	5	1.25	
	Sub-Total-3	**241**	**60.25**	
	Grand Total	**524**	**131**	

Fig. 3.1: Sewage pumping stationsin Gujranwala city

3.2 MAIN DRAINS IN GUJRANWALA

There are four main open drains in Gujranwala city (Fig. 3.2), in which the wastewater is disposed by the seweage DS. These main drains are listed below

1. Main Drain
2. QilaMian Singh Drain
3. Mir Shikara Drain
4. Jinnah Road Drain.
5. AduRai Drain.

A layout plan of these drains are shown in Fig. 3.2

From Table 3.1 the estimated present flow in each open drain can be evaluated. The average present flows in Table 3.1 have been calculated by dividing the pumping capacity by 4, by assuming a peak factor of 4. The flow from DS at serial number 10 to 16 is divided into two drains viz: (1) QilaMian Singh Minor and (2) Mir Shikaran. At this early stage of inception, it is assumed that 67% of the total flow (60.24 cusec) from these disposal station goes to QilaMian while rest i.e. 33% goes to Mir Shikara. Thus estimated flow in each drain is shown in Table 3.2.

Table 3.2: Present estimated flow in drains

Sr. No.	Drain	Estimated Present Flow (2013) (cusec)
1	Main Drain	56.25
2	Jinnah Road Drain	12
3	QilaMian Singh Minor	40.37
4	Mir Shikaran	19.88
5	AduRai	2.5
	Total	**131**

The above flows are estimated present flows derived from the pumping capacity of each sewage pumping station that contribute wastewater to a specific drain. Detailed calculations are shown in Table 5.2.

Fig. 3.2: Layout of main drains in Gujranwala city

3.3 FINAL DISPOSAL OF WASTEWATER FROM THE CITY

The wastewater from the 17 DS is pumped to the five main drains which lead the wastewater to the final disposal point i.e. Qadir Abad Balliki Link Canal.

Chapter- 4

DISCUSSION WITH WASA OFFICIALS

4.1 VISIT TO WASA OFFICE GUJRANWALA

A preliminary visit, before signing of the agreement, was made on 21.02.2013 to WASA office Gujranwala. The purpose was to introduce the project team. A preliminary proposal for wastewater treatment plant (WWTP) marked on Google image was discussed with MD WASA. The objective was to obtain WASA views. Thereafter a detailed discussion about the land requirement and availability and the possible strategy to carry out the wastewater treatment was discussed. The MD WASA showed his concerns regarding land availability and gave some valuable suggestions as broad policy guideline for the project. Following persons were present in the meeting.

From WASA Gujranwala

1.	**Mr. Khalid Bashir Butt**	MD WASA **(In Chair)**
2.	Mr. FidaHussain	Director Engineering WASA
3.	Mr. TalibHussain	Deputy Director Nandipur Town WASA
4.	Mr. AtharNadeem	SDO Aroop Zone WASA

From Jers Engineering Consultant (JEC)

1.	Mr. Shahid A. Siddiqui	Director Engineering (Jers)
2.	Dr. Sajjad Hussain	Technical Advisor (Jers)
3.	Mr. FarooghMarghoob	Director Administration (Jers)
4.	Mr. Faisal Butt	Senior Design Engineer (Jers)

4.2 SUMMARY OF DISCUSSION WITH WASA

1. MD WASA proposed that instead of one combined WWTP for the city, it is better to have multipleWWTP outside the boundaries of bypass road.It was suggested

by MD WASA that acquisition of land and arrangement of funds for one combined WWTP would be difficult as large piece of land and huge funds would be required. It is therefore preferable that the objective of wastewater treatment may be achieved through a staged process of having multiple WWTPs. It would help to divide the whole project into smaller portions and prioritize. This would also help to break down the entire cost and take up prioritized works one by one depending upon the availability of funds. Moreover, land requirements for multiple WWTPs would be less and it will facilitate the land acquisition process. Moreover, it would also be difficult to acquire one large piece of land at one place. All the participants agreed to this concept. Mr. Shahid told that Jers would revise its proposal of one WWTP and split it into multiple WWTPs for the city.

2. MD WASA pointed out that there are 5main drains named MianSingh Minor, Mir Shikaran Drain, Main Drain, Jinnah Road Drain and AdhuRai Drain. He said that AdhuRai Drain may be taken up at a later stage.
3. The above drains were prioritized based on the quantum of flow in them as given below
   1) MianSingh Minor and Mir Shikaran Drain
   2) Main drain
   3) Jinnah Road drain
   4) AdhuRai drain
4. The first preference may be given to Miansingh minor and Mir Shikaran Drain for treatment.
5. The second preference be given to Main drain and third preference to Jinnah Road drain for treatment.
6. The land availability near AdhuRai drain seems to be difficult, so only proposal will be given for its treatment.
7. MD WASA indicated possible land areas for WWTP, which were marked on the Google map for further investigations by the consultants.
8. The client also suggested the consultant to visit model Disposal station at Rajkot.
9. Singing of the agreement was requested to WASA to formally start the work.

Chapter- 5

PRELIMINARY PROPOSAL FOR WWTP/S

5.1 COMBINED WWTP FOR THE ENTIRE CITY

At the time of visit of consultant to WASA office, a proposal of one combined WWTP for the entire city was marked on the Google map. This proposal was based on Waste Stabilization Ponds. The concept was that the wastewater of the whole city would be collected at one place for treatment. The proposed site and its area is shown in Fig. 5.1. However, this idea has now been dropped due the reasons mentioned in chapter 4 above.

5.2 MULTIPLE WWTPs FOR THE CITY

Multiple WWTPs can be planned based on the layout of main drains of the city. Following proposal are presented for further discussion and feedback from WASA. Initially, the sites of WWTPs are proposed based on availability of land from Google images. Presently, the city area is thickly populated and land availability for WWTPs is non-existent within the city. Furthermore, WWTPs within city area is also environmentally undesirable. Therefore, the WWTPs would have to be located outside the city boundary.

Based on the discussion with WASA officials, following WWTPs are proposed for the city of Gujranwala. The following list is arranged with respect to the priority suggested by WASA.

1. WWTP-QilaMian Singh
2. WWTP-Main Drain and Jinnah Raod (combined)
3. WWTP-Mir Shikaran
4. WWTP-AduRai Drain

5.3 ESTIMATION OF POPULATION AND WASTEWATER FLOWS

Present wastewater flows are shown in Table 3.2. The flow estimation for future i.e. year 2038 has been estimated and shown in table 5.1. The basis of calculation presented in the table has been provided in the notes given below the table.

Table 5.1: Future population and flow forecasting

Sr.No.	Drain	Contributing Disposal Stations	Present Sewage pumping capacity (2013) (Cusecs)	Present Average sewage flow (2013)		Pop (2013)	Pop (2038)	Flow (2038)	
				cusec	m³/day			m³/day	cusec
1	Main Drain	Khayali	70	17.5					
		Sarfaraz Colony	10	2.5					
		Peoples Colony	80	20					
		ittefaq Colony	5	1.25					
		PMU	50	12.5					
		Ferozwala	5	1.25					
		Abu Bakar Park	5	1.25					
		Total	**225**	**56.25**	**137,587**	**764,375**	**1,208,738**	**217,573**	**89**
2	Jinnah Road Drain	NowsheraSansi Road	40	10					
		Mughalpura	8	2					
		Total	**48**	**12**	**29,352**	**163,067**	**257,864**	**46,416**	**19**
	Complete Flow (QilaMian Singh Minor + Mir Shikaran Drain)	Smanabad	10	2.5					
		Rajkot	120	30					
		Garjakh	28	7					
		AlamChowk 1	8	2					
		AlamChowk 2	30	7.5					
		Nowshera Road	40	10					
		Zahid Colony	5	1.25					
		Total	**241**	**60.25**					

Sr.No.	Drain	Contributing Disposal Stations	Present Sewage pumping capacity (2013) (Cusecs)	Present Average sewage flow (2013)		Pop (2013)	Pop (2038)	Flow (2038)	
				cusec	m^3/day			m^3/day	cusec
3	QilaMian Singh Minor	Total	161.47	40.37	98,738	548,549	867,444	156,140	64
4	Mir Shikaran Drain	Total	79.53	19.88	48,632	270,181	427,249	76,905	31
5	AduRai			2.5	6,115	33,972	53,722	9,670	4
Grand Total			**514**	**129**					**207**

Notes on Table 5.1

1. Present population (2013) has been calculated from the present average flows. For this purpose WASA design criteria was used. As per WASA criteria per capita water consumption is 50 GPCD or 225 LPCD. If 80% of it is taken as sewage flow then per capita sewage flow will be 225x80%=180 LPCD or 0.18 m^3/day. By dividing this per capita sewage flow with present average flow, population of 2013 has been evaluated.
2. For evaluating population of 2038, the population of 2013 was taken as basis. For calculating 2038 population, a growth rate of 1.85% was adopted (for basis of adopted growth rate please see section 6.7). Geometric growth rate formula is used for population projection as given below.

$$P_f = P_p (1+x)^n \qquad \text{(Equation-1)}$$

Where;
Pf= Future population
P_p = present population
x = growth rate
n = no. of years in future for which population is required.

Fig. 5.1: Location of combined WWTP

5.4 LOCATION OF ALL MULTIPLE WWTPS

All the WWTPs are shown in the same Figure i.e. Fig. 5.2.

5.4.1 WWTP Main Drain and Jinnah Road Drain

Jinnah Road Drain meets the Main Drain as shown in Fig. 5.2. Therefore, a combined WWTP is proposed for both these drains. The design flow (2038) for these drains are 89 cusec (217,573 m^3/day) and 19 cusec (46,416 m^3/day), respectively as shown in Table 5.1. The combined flow is 108 cusec (263,989). The location of WWTP is shown in Fig. 5.3.

5.4.2 WWTP-QilaMian Singh Drain

QilaMian Singh Drain is a major drain. The estimated design flow (2038) is 64 cusec(156,140 m^3/day). The proposed location of WWTP QilaMian Singh Drain is shown in Fig. 5.4.

5.4.3 WWTP-Mir Shikaran

The estimated design flow (2038) of Mir Shikara Drain is 31 cusec(76,905 m^3/day). Two sites are available for this WWTP. These two options are shown in Fig. 5.5. Location on the upstream side is captioned as Option-1 and location on the downstream side is captioned as option-2.

5.4.4 WWTP-AduRai Drain

The estimated design flow (2038) for AduRai drain is 4 cusec (9,670 m^3/day). The proposed location of WWTP AduRai Drain is shown in Fig. 5.6.

Fig. 5.2: Location of all multiple WWTP

Fig. 5.3: Location of WWTP Main Drain and Jinnah Drain

Fig. 5.4: Location of WWTP QilaMian Singh Drain

Fig. 5.5: Location of WWTP Mir Shikara Drain

Fig. 5.6: Location of WWTP AduRai Drain

Chapter- 6

TECHNOLOGICAL OPTIONS FOR WASTEWATER TREATMENT IN GUJRANWALA

6.1 GENERAL

Different treatment technologies are available for wastewater treatment in Gujranwala. These range from relatively simple low-cost options like Waste Stabilization Ponds (WSP) to highly mechanized and costly systems, for instance the Activated Sludge Process (ASP). The choice between the various options depends upon the following factors: (1) the effluent standards to be achieved; (2) the capital and maintenance cost of each option and (3) institutional capacity available to run an option. A brief comparison of various treatment options, which can be used in Gujranwala, is given below.

1. Activated sludge process (ASP)
2. Upflow Anaerobic Sludge Blanket (UASB)
3. Biological Trickling Filter Process (BTF)
4. Oxidation ditch (OD)
5. Aerated lagoons (AE)
6. Wastewater Stabilization Ponds (WSP)

A brief discussion and comparison of each options is given in the following sections

6.2 ACTIVATED SLUDGE PROCESS (ASP)

In this process, a mixture of WASTEWATERand ACTIVATED SLUDGE is agitated and aerated in an aeration tank. Bacteria present in activated sludge aerobically metabolize the organic matter present in the influent. The organic matter is oxidized to CO_2, H_2O, NH_3etc and a portion of it is converted into new bacterial cells. The activated sludge is subsequently separated from MIXED LIQUOR by gravity in the secondary sedimentation tank and either wastedor returned to the aeration tank as needed.A concept diagram of ASP is shown in Fig. 6.1.

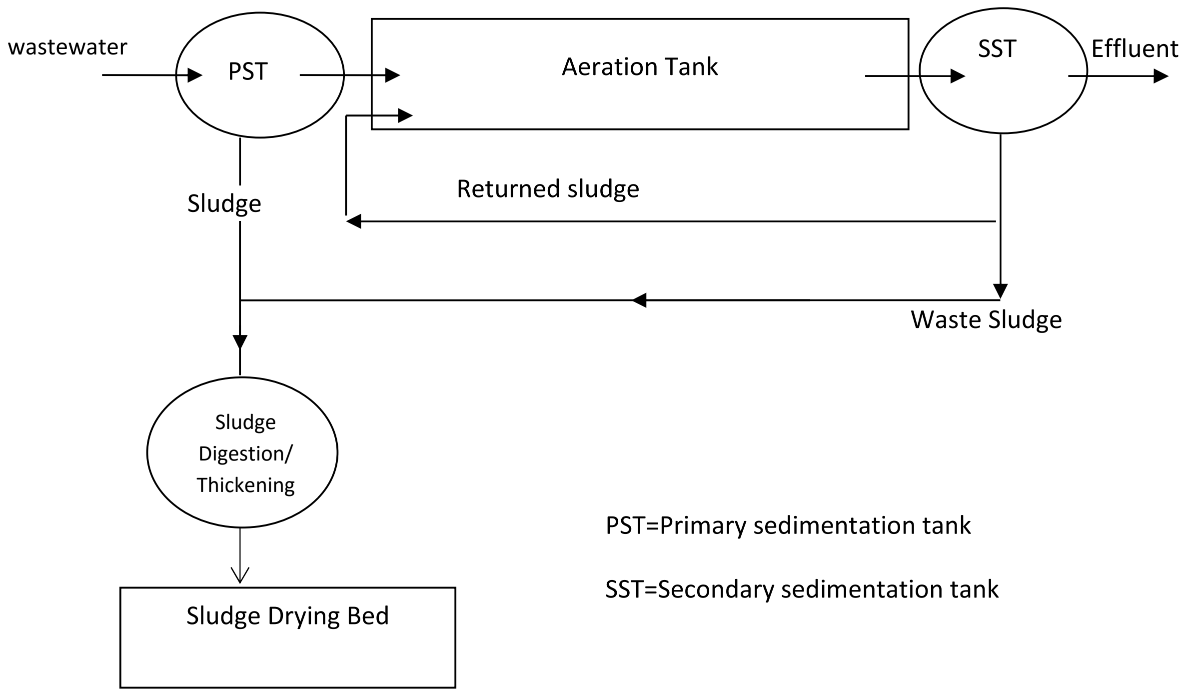

Fig. 6.1: Concept diagram of ASP

No doubt the land requirement for Activated Sludge Process (ASP) is small as compared to Waste Stabilization Ponds (WSP) but the cost, both capital and operation, is much higher than that of the WSP. The major drawback of the ASP is that the process involved is very sensitive and any interruption in electricity or any other slackness in process control may disrupt the process completely. If this happens, then the intended efficiency of the system cannot be achieved. At worst, sewage will be discharged without any treatment. Since the ASP is a biological process and microorganisms treat the wastewater, therefore, it is a matter of great concern that the startup time (the time when beneficial microorganisms grow to sufficient number to treat the wastewater) for the ASP is 1 to 1.5 months. If the process is disrupted within this period owing to long electricity load shedding hours, which is a normal routine in Pakistan, it is suspected that the plant may never achieve its full treatment capacity.

ASP is electricity intensive system. Furthermore, it requires import of equipment (aerators etc) with no local expertise to repair them. The above factors will make the O & M cost prohibitively high and unaffordable under local conditions. Skilled labour required to run the system is also not locally available. No institutional capacity exists in Public Health Engineering Department (PHED), Punjab, to run such a complicated system. All the above factors do not support the use of ASP in Gujranwala.

6.3 UPFLOW ANAEROBIC SLUDGE BLANKET (UASB)

It is a special kind of anaerobic treatment rector, which consist of empty tank. The wastewater enters from suitable spaced inlet to tank and with upward flow direction through a sludge bed where microbes are present in the form of ganules. These granules come into contact with substrates. Granules, have high sedimentation velocity and show resistance in wash out from system even at high flow or hydraulic loads. Anaerobic degradation of wastewater is typically responsible for the generation of biogas (mixture of CO_2 and CH_4). The gas bubble move upward and create hydraulic turbulence that provide automatic mixing without any mechanical mean. At the top of reactor, the treated water is separated from gas and sludge solids in three phase separator. The three phase separator is usually a cap with a settler located above it. Baffles are normally provided under the opening of gas cap to deflect produced gas towards gas-cap opening. A concept diagram of UASB is shown in Figure 6.2 (Saleh and Mehmood, 2003).[2]

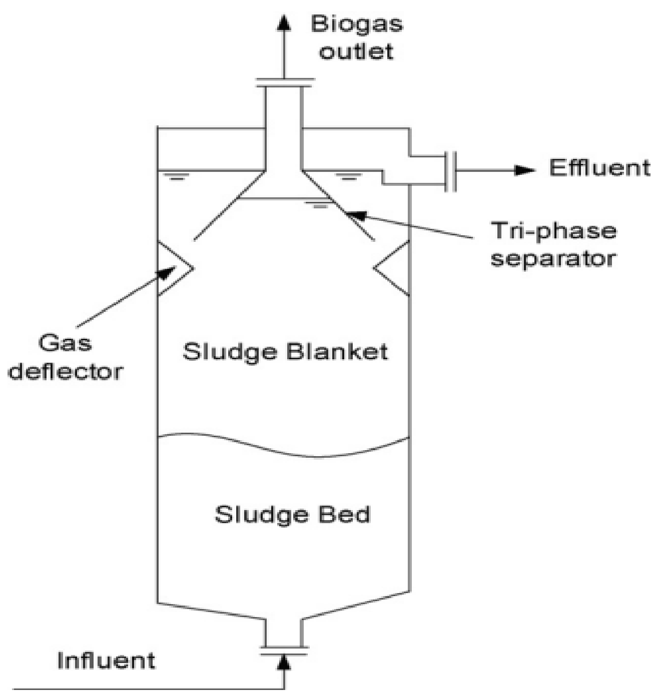

Fig. 6.2: Upward-flow anaerobic sludge bed (UASB) reactor

[2]Saleh and Mehmood(2003), UASB/EGSB Applications For Industrial Wastewater Treatment, Seventh International Water Technology Conference, Egypt, pp.1-3

6.4 BIOLOGICAL TRICKLING FILTER (BTF)

The use of TF began in 1901. Much work went into developing the empirical design approach, presently in use, by the US Army Corps of Engineers during the second world war. T.F utilizes a relatively porous bacterial growth medium like ROCK or FORMED PLASTIC SHAPES. Wastewater is applied to the surface and percolates through the filter, flowing over the biological growth in a thin film.Bacterial Growth occurs upon the surface while oxygen is provided by air diffusion through void spaces.Nutrients, oxygen and organic matter are transferred to the fixed water layer and from there to bacteria.Waste products are transferred to the moving water layer, primarily by diffusion.TF has no power requirements except for pumping the wastewater as and when required. A concept diagram of BTF is shown in Fig. 6.3.

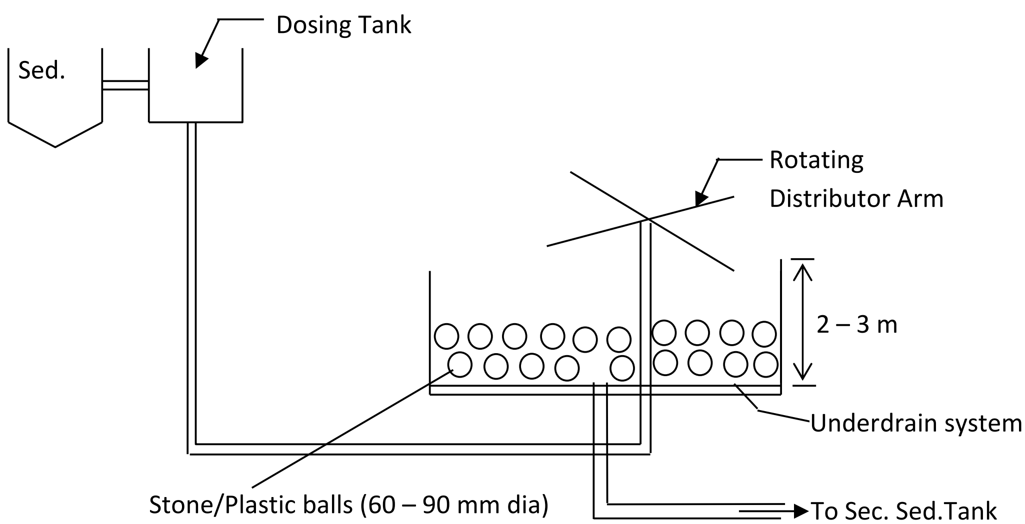

Fig. 6.3: Concept diagram of BTF

Process complexity of TF is less as compared to ASP and AL. However, the capital cost, although less than ASP is more than AL. The odour and fly problem are major issues with conventional TF using rock media.

6.5 OXIDATION DITCH (OD)

The oxidation ditch is a modified form of ASP in which aeration of wastewater is done for a much longer time than in ASP. Aeration time in a normal ASP lies in a range of 6-10 hours while aeration time in OD lies in a range of 24-36 hours. For this reason, oxidation ditch technology is referred to as a form of extended aeration technology. It is usually used for smaller discharges. Its disadvantages are the same as that of ASP stated in section 6.2. A concept diagram of OD is shown in Fig. 6.4.

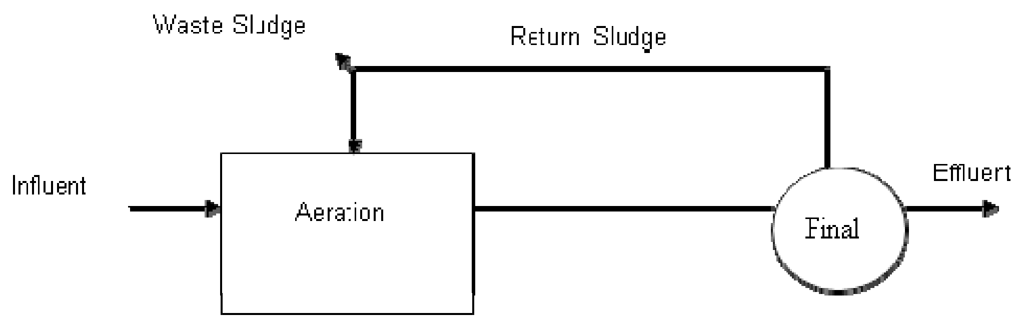

Fig. 6.4: Concept diagram of OD

6.6 AERATED LAGOON (AL)

It is a treatment process which actually developed from overloaded facultative ponds by introducing aerator to improve the treatment efficiency. It requires a relatively large area as compared with ASPs and is electricity intensive. It is less costly than ASP but requires high O&M cost. The disadvantages are the same as narrated above for the ASP in section 3.2. A concept diagram of AL is shown in Fig. 6.5.

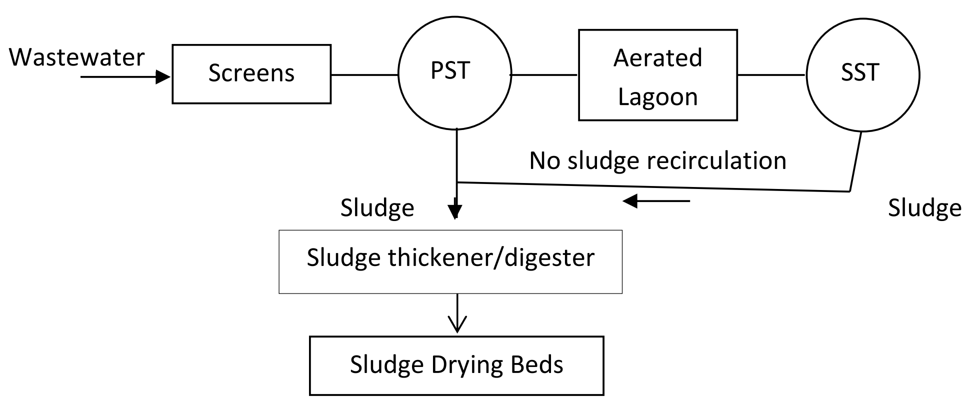

Fig. 6.5: Concept diagram of AL

6.7 WASTEWATER STABILIZATION PONDS (WSP)

WSP is a relatively shallow body of water contained in an earthen/lined basin of controlled shape which is designed for the purpose of treating wastewater. It uses natural processes for the treatment of wastewater. No electricity is used during the process.

WSP is the most robust, cheap and tested technology for the treatment of wastewater. It works best in hot climates and is being used in 60 different countries of the world (Gloyna, 1971)[3].

Statistics show that one third of all the wastewater treatment plants in USA are based on WSP (Mara, 1997)[4]. There are 7000 WSP in the USA (USEPA, 1983)[5] alone. Take the example of other technologically advanced countries of Europe. There are 2500 WSPs in France and 2000 WSPs in Germany (Mostafaei, 1996)[6]. In Israel, due to water scarcity, WSP is the first choice of treatment because the treated water in WSPs is the most suitable for irrigation purposes (Mara, 1998)[7].

The WSP have a number of benefits which make it the best choice for developing countries and these are as follows:

- **i.** Have almost negligible power requirements and therefore most suitable for Pakistan.
- **ii.** Low in construction and operational cost
- **iii.** No skilled man power is required to run the system as is required in ASP, OD etc
- **iv.** Minimum sludge handling problems. However needs desludging after 5-10 years.
- **v.** Can produce effluent fit for irrigation(Shuval, 1990)[8]

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[3]**Gloyna**, E. F. (1971), Waste Stabilization Ponds, WHO, Geneva.

[4]**Mara**, D. (1997), Wastewater stabilization ponds design manual for India.Page 2.

[5]**US EPA** (1983), Design Manual of Municipal Wastewater Stabilization Ponds, EPA-625/1-83-015

[6]**Mostafaei**, A. (1996), Stabilization ponds in Europe, Journal of Water and Environment, No. 21, pp 38-40

[7]**Mara**, D. (1998), Wastewater stabilization pond design manual for Mediterranean countries, page 31.

[8]**Shuval**, H. I. (1990), Wastewater irrigation in developing countries, health effects and technical solutions, World Bank technical paper No. 51 (World Bank water and sanitation program), page 27

vi. Does not require import of equipments/spares

vii. Most suitable in hot climates.

The only disadvantage of WSP is its larger area requirement.

6.8 Population forecasting

For future population projection, the census of 1998 has been adopted as the base year. The growth rate, from year 1998 to 2013 has been adopted as 2.15% per annum. The demographic trends in Pakistan has been well document. The Fig. 6.1 shows the annual growth rates over past few decades. It is evident from the graph that the growth rates touched its peak in 1980s and thereafter decreased. It appears to have been stabilized after 2010 or the sharp decline has stopped. It may further decrease, however, sufficient data is not available for further projection.

Keeping in view the trend in Fig. 6.1, a growth rate of 1.85% has been adopted after year 2013. The population projections and corresponding sewage flows are shown inTable 6.1.

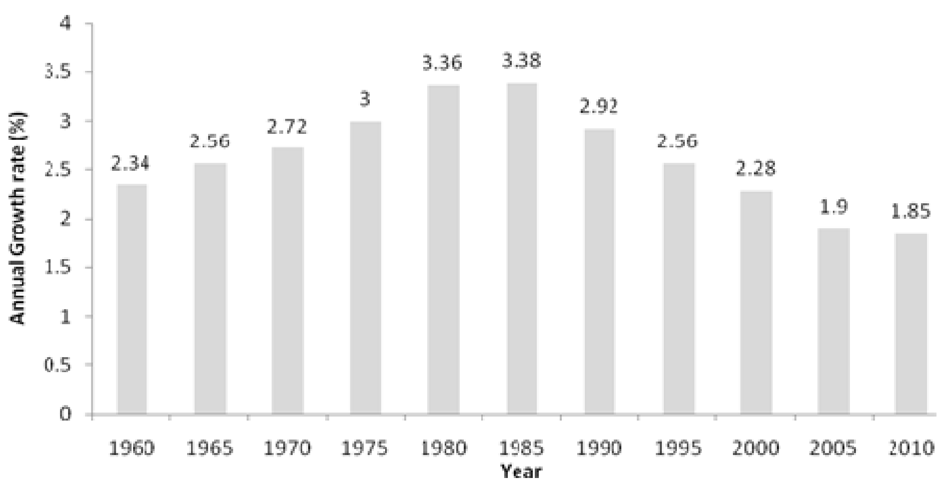


Fig. 6.6: Demographic trends in Pakistan

(Source: Data from World Bank)[9]

[9]http://www.google.com.pk/publicdata/explore?ds=d5bncppjof8f9_&met_y=sp_pop_grow&idim=country:PAK&dl=en&hl=en&q=current%20population%20growth%20rate%20of%20pakistan

Table 6.1: Population projection and sewage flows for Gujranwala city

Year	Growth rate (%)	Population	Per capita average wastewater flow (m^3/day)	Total average flow (m^3/day)	Total average flow (cusec)
1998	-	1,132,509	0.18*	203,852	83
2013	2.15	1,661,889	0.18	299,140	122
2024	1.85	2,033,169	0.18	365,970	149
2038	1.85	2,628,016	0.18	473,043	193

**0.18=50 GPCDx4.5Literx80%/1000*

By comparison of Table 6.1 and 5.1, it can be seen that the flow projections from pumping capacity lies quite close to flow projection from population data. From pumping capacity, the projected design flow (2038) is 207 cusec while from population figures the design flow (2038) is 193 cusec. The difference is only 6% $[(207 - 193) \times 100]/207$, which is negligible.

6.9 COMPARISON OF DIFFERENT OPTIONS

A comparison of above treatment options has been made with respect to the followings four parameters:

1. Construction cost
2. Total operational cost including electricity
3. Cost of electricity
4. Land requirements

For the above comparison the following literature was consulted:

1. *Preparatory study on Lahore water supply, sewerage and drainage improvement project in Islamic Republic of Pakistan (July, 2009), Japan International Cooperation Agency **(JICA)**, NJS Consultants and CTI Engineering International.*
2. *Guidelines and explanation for Planning and Design of Sewage facilities (2001 edition) P-341" (**Japan Sewage Work Association**)*
3. ***Metcalf & Eddy** "Wastewater Engineering-Forth Edition" , pp-851 Example 8-15, Mcgraw-Hill, USA.*

Using the information given in above references, an excel sheet was developed to forecast the above parameter from population, per capital sewage flow. The values of parameters compared below are indicative and can be used for comparison. However, there may be minor changes when actual design will be undertaken. The results for Gujranwala for the design population of 2038 are given below.

6.9.1 Comparison of construction cost

A comparison of the construction cost of above 6 treatment technologies will be made in this section. The basis of comparison is given in Table 6.2. The cost basis give indicative cost for the purpose of comparison among different technologies.

Table 6.2: Basis of construction cost for different treatment technologies (Source: JICA, 2009)

Treatment Method	Conatruction Unit Cost ($Rs/m^3/d$)
Waste Stabilization Ponds	7100
Aerated Lagoons	8391
Trickling Filters	15,492
UASB	19,149
Oxidation Ditches	15,492
Activated Sludge	21,516

Based on Table 6.2, a comparison of capital cost, to treat a wastewater discharge of 473,043 m^3/day (193 cusec) for year 2038, from Gujranwala city is shown in Fig. 6.7

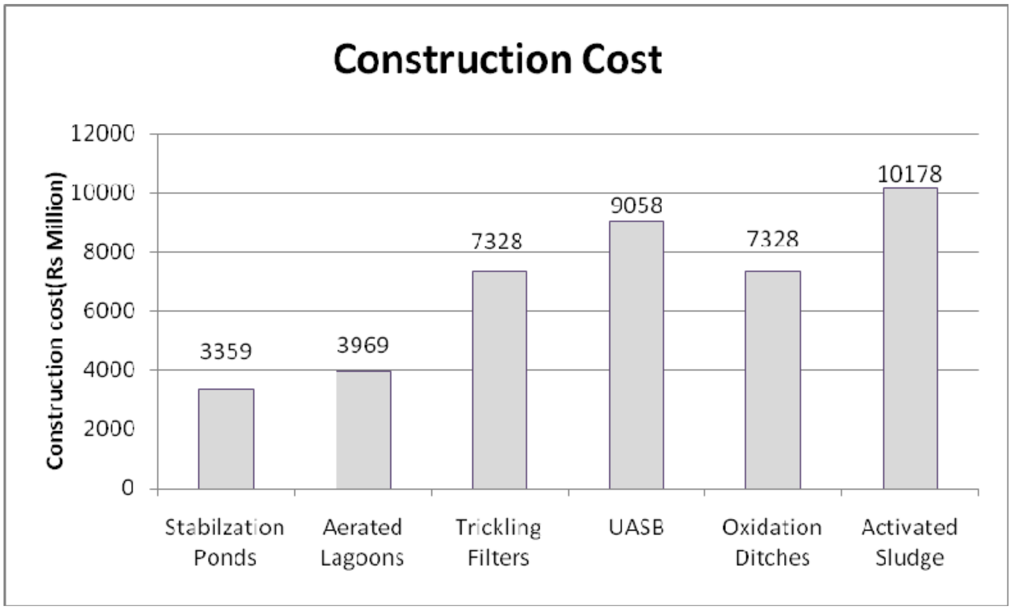


Fig. 6.7: Comparison of construction cost

6.9.2 Comparison of total operation cost including electricity

The basis of total operation cost including electricity is given in Table 6.3.

Table 6.3: Basis for total operational cost including electricity
(Source: JICA, 2009)

Treatment Method	Total operational cost including electricity (Rs/m3/yr)
Waste Stabilization Ponds	56
Aerated Lagoons	1409
Trickling Filters	673
UASB	448
Oxidation Ditches	2281
Activated Sludge	1826

Based on Table 6.3, a comparison of total operational cost including electricity, to treat a wastewater discharge of 473,043 m^3/day (193 cusec) for year 2038, from Gujranwala city is shown in Fig. 6.8.

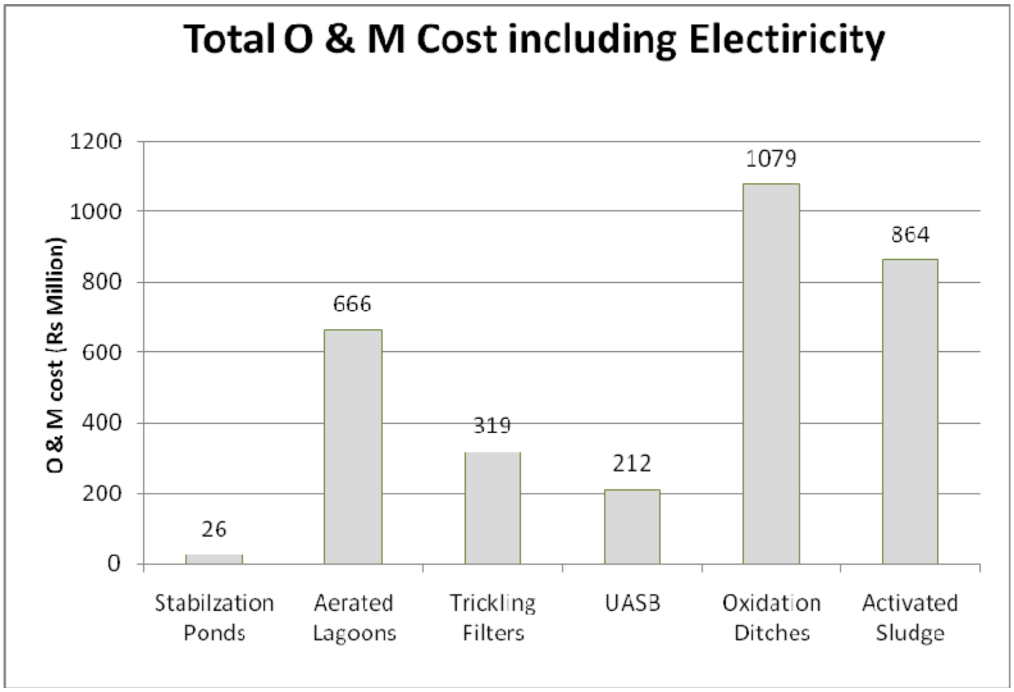


Fig. 6.8: Comparison of total annual operational cost including electricity

6.9.3 Comparison of electricity cost to run process

The basis of electricity cost to run different treatment technologies is given in Table 6.4.

Table 6.4: Basis of electricity cost to run a technology (Source: JICA, 2009)

Treatment Method	Electricity Unit Cost (Rs/m3/yr)
Waste Stabilization Ponds	0
Aerated Lagoons	1290
Trickling Filters	568
UASB	396
Oxidation Ditches	2202
Activated Sludge	1721

Yearly consumption of electricity to run the process in each technology for wastewater flow of 473,043 m^3/day (193 cusec)is shown in Fig. 6.9.

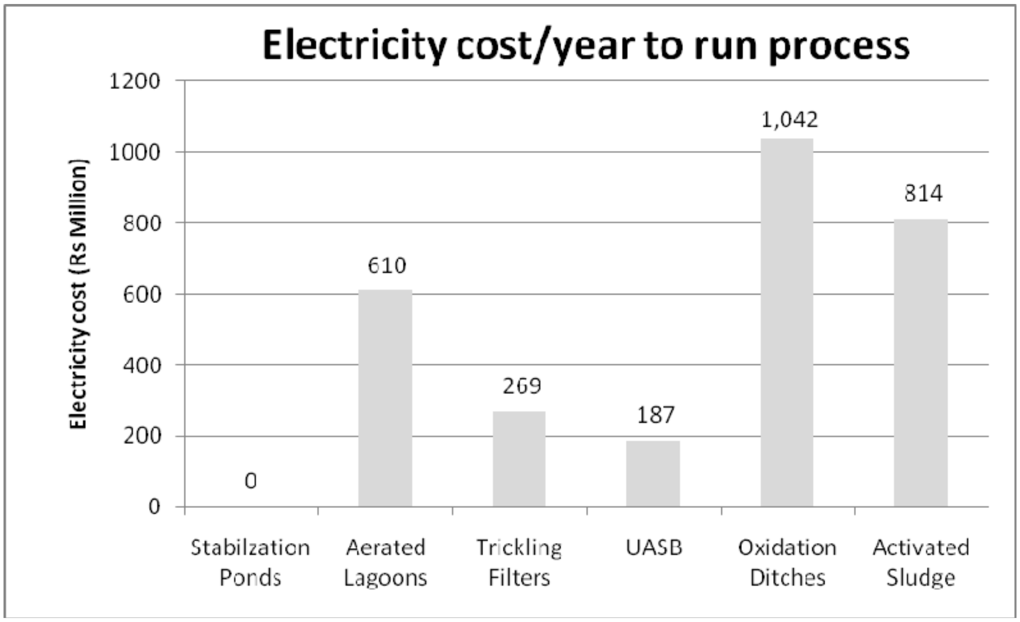


Fig. 6.9: Comparison of annual electricity consumption in different technologies

6.9.4 Comparison of land requirements.

The basis for the calculation of land requirement for different treatment technologies has been presented in Table 6.5.

Table 6.5: Basis of land requirement for different technologies

(Source: Kevin Taylor, 2009;World Bank Consultant)

Option	Land/person (m^2)
Waste Stabilization Pond	1.6
Aerated Lagoon	0.9
Trickling Filter	0.5
UASB	0.3
Oxidation Ditch	0.4
Activated Sludge	0.3

Land requirements for each technology for wastewater flow of 473,043 m^3/day (193 cusec) with a population of 2,628,016 persons are given in Fig. 6.10.

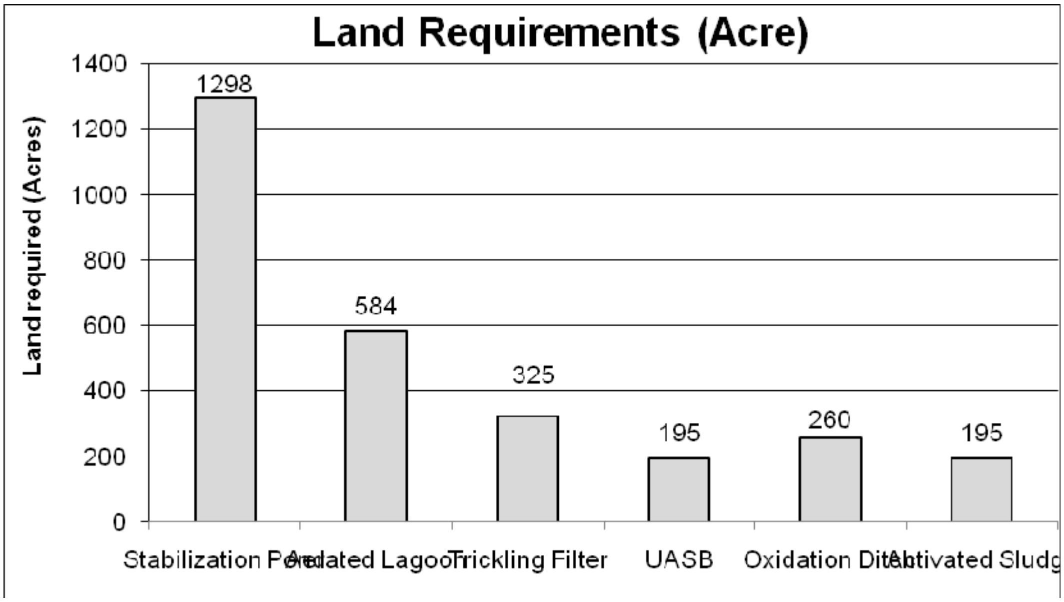


Fig. 6.10: Comparison of land requirements

6.10 Summary Table

A summary of above comparison is presented in Table 6.6

Table 6.6: Comparison table of treatment technologies

	ASP	AL	TF	UASB	OD	WSP
Construction cost (Rs Million)	10178	3969	7328	9058	7328	3359
Total annual O&M cost including electricity (Rs Million)	864	666	319	212	1077	864
Electricity cost per year (Rs Million)	814	610	269	187	1042	814
Land requirement (Hectare)	195	584	325	195	260	1298
Conclusion	Based on construction and operational cost WSP is most suitable					

6.11 RECOMMENDED TECHNOLOGY

Based upon the above analysis, it appears that WSPs may prove to be the best for Gujranwala with respect to construction, operational and electricity cost. The key point here is that WSPs are simple and it is doubtful whether higher technology solutions would work.

Chapter- 7

BRIEF INTRODUCTION ON WASTE STABILIZATION PONDS

7.1 BRIEF HISTORY

Ponds have been used for centuries to store and treat animal and house-hold wastes. However, with in the last few decades, specific design criteria have been developed in terms of volumetric requirements, organic loading rates and detention times.

Up to 1940s little engineering or research went into the construction of WSPs, some of which failed. Field studies were started in the decade of 1940-50 to develop a rational design criteria. By 1962, there were 1647 stabilization ponds in use in USA for the treatment of municipal wastes.WSPs are now in use in more than 50 countries including USA, Canada, Australiaand Germany. WSP employ natural processes for the treatment of wastewater.

7.2 TYPES OF WSP

WSPs are divided into three types viz:

1. Anaerobic ponds
2. Facultative ponds
3. Maturation ponds

7.2.1 Anaerobic ponds (AP)

Anaerobic ponds (APs) are fully anaerobic throughout their depth. They provide initial treatment to the wastewater. Suspended solids load in wastewater is mostly removed in AP and settle as sludge at the bottom where it undergoes anaerobic decomposition. The efficiency depends upon the ambient temperature. Approximately around 50% of BOD reduction could take place in APs. They are generally 3-4.5 m deep with side slopes of 1 Vertical: 3 Horizontal. The detention time may vary from 1-5 days. Effluent from AP needs further treatment. They are normally desludged after a period of 5 years. A schematic diagram of AP is shown in Fig. 7.1.

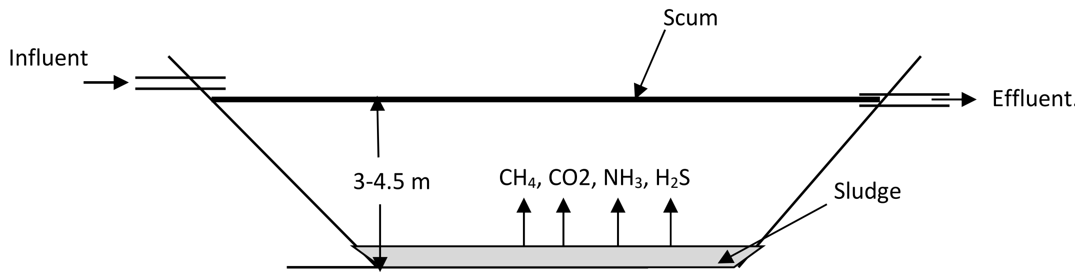


Fig. 7.1: Schematic diagram of AP

7.2.2 Facultative ponds (FP)

Facultative pond (FP) usually come after AP and used to further polish the effluent from AP. The upper layer of FP is aerobid, the middle layer is facultative in which both aerobic and anaerobic bacteria treat the wastewater. The bottom lay is anerobic. The detention time is FP usually varies from 5-12 days and its depth varies from 1.5 to 2 m. FP brings down the BOD of wastewater to such levels which are safe for disposal in the water bodies. A schematic diagram of FP is shown in Fig. 7.2.

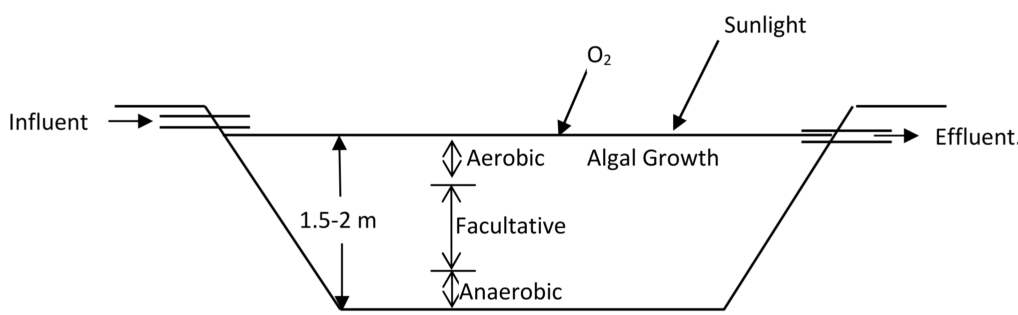
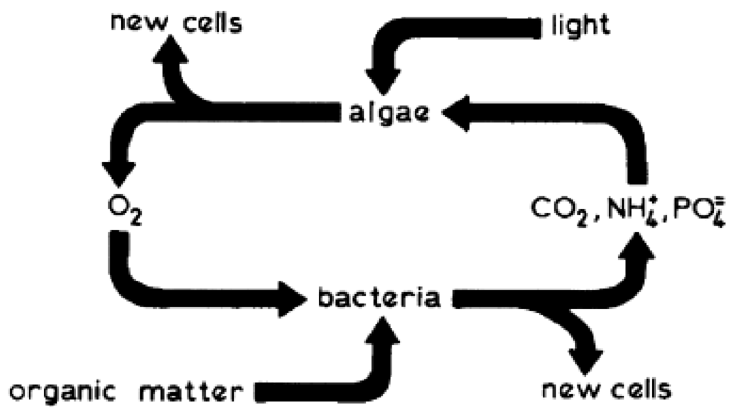


Fig. 7.2: Schematic diagram of FP

In aerobic zone of FP, bacteria and algae exist in a mutually beneficial or Symbiotic relationship. Algae produce O_2 during photosynthesis which is needed by bacteria to metabolize organic matter whereas bacteria release CO_2 and other inorganic matter like Nitrogen and Phosphorous which are needed by algae to grow and meet its food requirement. Hence under normal light conditions, the metabolic action of these two microbial groups complements each other.

A schematic of this Symbiosis is shown below.



Bacteria-Algae Symbiosis

BOD removal in facultative pond is usually in the range 70-80% based on unfiltered samples. However, BOD removal in FP can also be model with the help of following relationship[10].

$$\frac{L_e}{L_i} = \frac{1}{1+K\,t} \text{ ------------------ (Equation-1)}$$

Where

L_e	=	Effluent BOD from FP(mg/L)
L_i	=	Influent BOD to FP(mg/L)
K	=	Reaction rate constant, per day
		(Normally 0.2 to 0.3 per day for domestic sewage)
t	=	Detention time in FP, days

7.2.3 Maturation ponds

Maturation ponds (MP) are used to reduce the fecal coliform count in the wastewater. These are normally employed under two conditions: (1) when the wastewater is to be used for un-restricted irrigation (like watering vegetables and crops which are eaten raw i.e. without washing; e.g. salads etc) or (2) when the effluent standards restrict the fecal coliform count to a specific number for disposal in water bodies like beaches, where people come for recreation and bathing. Pakistan has no standards for fecal coliform count, hence MP are required. MP are fully aerobic throughout their

[10]**Eckenfelder**, W. W. (1970), Water pollution control, experimental conditions for process design, Jenkins book publishing, New York, USA. Page 159.

depth. Normally these are 1-1.5 m deep. A schematic diagram of MP is shown in Fig. 7.3.

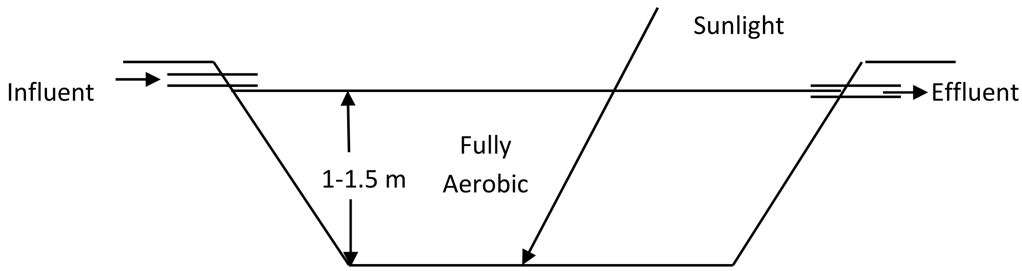


Fig. 7.3: Schematic diagram of MP

The fecal coliform removal in MP can be modeled by using the following relationship[11].

$$\frac{N_e}{N_i} = \frac{1}{1+kt} \qquad \text{------------------ (Equation-2)}$$

Where;

N_i = Number of coliform in the inflent/100mL (usually in a range of 10^7 – 10^8/100 mL in raw domestic wastewater)

N_e = Number of coliform in efflent/100mL

k = Bacterial die away constant,
(usually taken as 2.6 per day)

t = Detention time in MP, days

If more reduction in coliform is required than more than one maturation pond is provided in series. Under such condition all the maturation pond are equally sized and this is the most efficient configuration. If more than one pond is provided say “n” ponds then the denominator of equation-2 will become $[(1+kt_1)\ (1+kt_2)\ (1+kt_3)\ldots.(\ (1+kt_n)]^n$.

[11]**Mara**, D. (1997), Manual of waste stabilization ponds in India, Page 40.

7.3 DESIGN OF WSP

The design of WSP can be divided into two partviz:

1. Process design
2. Physical design

7.3.1 Process design

Process design includes the determination of volumetric requirements of ponds and their areas. Laid down criteria is used for process design. A detailed discussion on the design criteria is presented in the next chapter.

7.3.2 Physical design

Physical design of the WSP include the following things

- Decision regarding actual pond dimensions consistent with the available site
- Correction location of the pond inlet and outlet to avoid short circuiting. Inlets and outlets are generally pipes of appropriate diameter.
- Geotechnical aspects of WSP are very important. Coefficient of permeability to decide about the lining of WSP and other soil investigation for correct embankment design play an important role in the successful operation of WSP.
- By-pass pipes, security fencing and notices are generally required.

The physical design of WSP must be carefully done; it is at least as important as process design and can significantly affect the treatment efficiency.

7.4 OPERATION AND MAINTENANCE OF WSP

7.4.1 Start-up procedure

Pond systems should preferably be commissioned at the beginning of the hot season so as to establish as quickly as possible the necessary microbial populations to effect waste stabilization. Prior to commissioning, all ponds must be free from vegetation.

Before discharging water into the A.P, it should be seeded with sludge, for example, from local septic tanks. As much sludge should be put at the bottom of the A.P as is possible and then wastewater should be released into the ponds. Initially odour will be released from the ponds as the bacteria will start functioning. Odour problems will decrease gradually with the passage of time.

During the time, the A.P is being filled, fill the F.P with freshwater (from a river, lake etc) so as to permit the gradual development of the algal and heterotrophic bacterial populations. If freshwater is unavailable, facultative ponds should be filled with raw sewage and left for three to four weeks to allow the microbial population to develop; a small amount of odour release is inevitable during the period.

7.4.2 Routine maintenance

The maintenance requirements of ponds are very simple, but they must be carried out regularly. Otherwise, there will be serious odour, fly and mosquito nuisance. Maintenance requirements and responsibilities must therefore be clearly defined at the design stage so as to avoid problems later. Routine maintenance tasks are as follows:

A. Cutting the grass on the embankments and removing it so that it does not fall into the pond (this is necessary to prevent the formation of mosquito-breeding habitats).
B. Removal of floating scum from the surface of facultative pond (this is required to maximize photosynthesis and surface re-aeration and prevent fly and mosquito breeding);
C. spraying the scum on anaerobic ponds (which should not be removed as it aids the treatment process), as necessary, with clean water or pond effluent, to prevent fly breeding;
D. removal of any accumulated solids in the inlets and outlets;
E. repair of any damage to the embankments caused by rodents, rabbits or other animals; and
F. Repair of any damage to external fences and gates. The operators must be given precise instructions on the frequency at which these tasks should be done, and their work must be constantly supervised. The supervisor/ foreman should be required to complete at monthly intervals a pond maintenance record sheet, an example of which is given below.

Pond maintenance record sheet

Pond location: ……………………………………………………………………………………….

Date and time: ……………………………………………………………………………………….

Weather conditions: ………………………………………………………………………………………..

Access road: State (vegetation, damage) maintenance carried out………………………….

……………………………………………………………………………………….

……………………………………………………………………………………….

Pond site: State, maintenance carried out………………………………….……………….

……………………………………………………………………………………….

……………………………………………………………………………………….

Visual inspection carried out:

POND NUMBER	1 (A.P)	2 (F.P)	OBSERVATION
Colour of water (green, brown/grey, pink/red, milky/clear			
Odour			
Scum, foam			
State of embankment (erosion, rodent damage, vegetation)			
Inlet and outlet (blockage)			

General observation, other maintenance carried out:……………..…………………………………………

……..

7.4.3 Staffing levels

In order that the routine O&M tasks can be properly done, WSP installations must be adequately staffed. The proposed system is quite simple and since no mechanical equipment is involved, therefore, minimum level of staffing is required as shown in the Table 7.1. Actual staffing level will be evaluated at detailed design of WSP for Gujranwala.

Table 7.1: Staffing requirement at WSP system

Staff level	Number
Sub Divisional Officer	1
Sub Engineer	1
Supervisor	1
Labourer	2
sweeper	3

7.4.4 Desludging and sludge disposal

A.P and F.P require desluding after a specified period of time. Anaerobic ponds require desludging when they are one third full of sludge or after a fixed period of time. The desludging period, for A.P, taken in this design is 3 years. Sludge removal in A.P can be achieved readily by using sludge pumps. These are commercially available. The sludge is discharged into either an adjacent earthen ditch (made especially for this purpose along the A.P prior to desludging) or a tanker to transport it to a site away from A.P where it will dry and dewater in the sun. The size of the earthen ditch, if used, for the WSP system at Gujranwala will be calculated at detail design stage.

The desludging period of F.P is normally 10 years. Same technique as used in A.P is employed for the desludgingof FP.

7.5 PROPOSAL FOR GUJRANWALA

Mostly WSP are used in combination for the treatment purpose. Keeping in view the environmental laws and enforced National Environmental Quality Standard (NEQS), a combination of AP followed by FP pond will meet the NEQS (Fig. 7.4). The ponds will be divided into multiple module of suitable number, keeping in view the wastewater flow and geometry of the site.

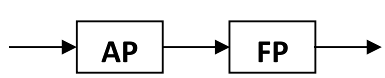


Fig. 7.4: Proposed WSP system for Gujranwala

Chapter- 8

DESIGN CRITERIA FOR WASTE STABILIZATION PONDS

8.1 GENERAL

Design criteria for WSPs in Gujranwala has been adopted from available International literature. Details are discussed in the following sections.

8.2 ANAEROBIC PONDS (AP)

AP is design on the basis of **volumetric loading** expressed as gm of BOD/m^3.day (g/m^3.day) based on the ambient temperature. The design criteria used for this report has been given in Table 8.1[12].

Table 8.1: Design criteria for A.P

Temperature °C	Permissible Loading (g/m^{3}d)	BOD removal (%)
<10	100	40
10 to 20	20T* – 100	2T + 20
20 to 25	10T +100	2T + 20
>25	350	70

**Where “T” is the mean temperature of coldest month in °C*

8.3 FACULTATIVE PONDS (FP)

Facultative ponds are designed design on the basis of **surface loading** and ambient temperature. The permissible loading is normally given in kg BOD/hectare.day (Kg/ha.day). The design equation used is that given by Arthur (1983)[13].

λs = 20T-60

where:

T = Mean Temperature of coldest month in °C

λs= Surface loading in Kg/hectare.day

[12] **Mara**, D. (1997), Wastewater stabilization ponds design manual for India. Page 34

[13] **Arthur**, J.P. (1983): Notes in the Design and Operation of Waste Stabilization Ponds in Warm Climates of Developing Countries, Washington,The World Bank Technical paper No. 7, page 19.

Chapter- 9

LAYOUT OF WWTP

9.1 FACILITIES IN WWTP

WWTP will comprise of the following facilities

1. Residences for staff
2. Laboratory
3. Office
4. Fencing with gate
5. Parking
6. Walkway
7. Lighting arrangements
8. WSP modules

9.2 LAYOUT OF FACILITIES

A layout of these facilities for a typical WWTP based on WSP is shown in Fig. 9.1.

As per our experience, the acquisition of land is perhaps the major hurdle in the timely completion of such projects. Therefore, this process may be initiated as soon as this Feasibility Report is submitted.

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Fig. 9.1: Typical layout of WWTP based on Waste Stabilization Ponds

Chapter- 10

LABORATORY DETAIL AND WASTEWATER TESTS AT WWTP

10.1 NEED OF LABORATORY

Establishment of a basic laboratory at the WWTP is essential to monitor the efficiency of the treatment system. Any upset in the system can be determined by routine tests conducted; both on influent and effluent wastewater. Test results help in gauging the satisfactory functioning of the plant and diagnosing any possible causes of malfunctioning. Hence a basic laboratory is proposed for this project.

10.2 NECESSARY WASTEWATER TESTS AT WWTP

In a basic laboratory following tests are recommended to be conducted on fortnightly basis.

1. pH
2. Total suspended solids (TSS)
3. Five day biochemical oxygen demand (BOD)
4. Chemical oxygen demand (COD)

A list of equipments and reagents required for the laboratory to conduct above tests is given below.

10.3 EQUIPMENT REQUIREMENT

Table 10.1 gives a list of equipment and their approximate cost for conducting the above tests.

Table 10.1: List and cost of equipment/reagent for laboratory

Sr No	Test and equipment/chemical	No.	Unit price (Rs)	Total cost (Rs)
1	**pH meter (Hatch Sension plus)**	1	165,000	165,000
	Beaker 1000 mL (pyrex)	5	825	4,125
	Beaker 500 mL (pyrex)	10	713	7,125
	Beaker 250 mL (pyrex)	10	563	5,625
	Sub-Total-1			**181,875**
2	**BOD test**			
	BOD Bottle	40	1,950	78,000

Sr No	Test and equipment/chemical	No.	Unit price (Rs)	Total cost (Rs)
	Compressor Pump for aeration,USA (used,2nd hand)	1	24,000	24,000
	Glassware			
	Pipette 10mL	10	413	4,125
	Beaker 500 mL (pyrex)	10	713	7,125
	Beaker 250 mL (pyrex)	10	563	5,625
	Burette 25mL (Germany)	5	3,750	18,750
	Titration Flask 250 mL (Pyrex)	10	563	5,625
	Reagents			
	i- Manganese Sulphate 1Kg (Merck/RDH)	1	2,850	2,850
	ii- Sodium Hydrooxide 1Kg (Fluka)	1	2,850	2,850
	iii- Sodium Iodide 1Kg (Fluka)	1	11,250	11,250
	iv- Sodium Azide 100g (Sigma)	1	7,500	7,500
	v- Conc. Sulfuric Acid 2.5L, 96% (Sigma)	1	9,750	9,750
	vi- Calcium Chloride 1Kg (RDH)	1	2,100	2,100
	vii- Magnesium Sulphate 1Kg (BDH)	1	3,000	3,000
	viii- Ferric Chloride 500g (Unicom)	1	3,000	3,000
	ix- Potasiummdihydrogen phosphate 1Kg (RDH)	1	3,300	3,300
	x- Potasiumm hydrogen phosphate 1Kg (RDH)	1	3,000	3,000
	xi- Starch 1Kg (BDH)	1	6,300	6,300
	xii- Sodium thiosulphate 1Kg (Fluka)	1	2,850	2,850
	Sub-Total-2			**201,000**
3	**COD test**			
	COD Apparatus (Lovibod)	1	435,000	435,000
	Glassware			
	Pipette 10mL	10	413	4,125
	Beaker 500 mL (pyrex)	5	713	3,563
	Beaker 250 mL (pyrex)	5	563	2,813
	Burette 25mL (Germany)	5	3,750	18,750
	Titration Flask 250 mL (Pyrex)	10	563	5,625
	Reagents			
	i- Ferrous ammonium sulphate 1Kg (RDH)	1	3,750	3,750
	ii- Mercury sulphate 100g (RDH)	1	9,000	9,000
	iii- Silver sulphate 5g (Permion)	1	4,500	4,500
	iv- Potassium dichromate 1KG (Merk)	1	7,350	7,350
	v- Conc. Sulfuric Acid 2.5L, 96% (Sigma)	1	9,750	9,750
	vi- Ferrous sulfate 1Kg (BDH)	1	4,200	4,200
	vii- 1,10-Phenanthroline monohydrate 1Kg (BDH)	1	3,750	3,750
	Sub-Total-3			**512,175**
4	**Total suspended solids test**			
	1.2 µm filter paper (whatman) 1pack	1	1,050	1,050
	Vacuum pump (Rocker)	1	63,000	63,000

Sr No	Test and equipment/chemical	No.	Unit price (Rs)	Total cost (Rs)
	Filtration assembly (Schott Duran)	1	24,000	24,000
	Drying oven 18 L capacity (China)	1	39,000	39,000
	Conical funnel	5	225	1,125
	Sub-Total-4			**128,175**
		Grand Total=		**1,023,225**

10.4 TEST PROCEDURES

For the training and information of laboratory staff, a brief details/testing procedure of each test to be conducted at the WWTP is given below. These have been adopted from the book "**Standard Methods for the Examination of Water and Wastewater (USA)**[14]" which is followed throughout the world as reference book for conducting tests on water and wastewater.

Test with testing procedure as per **Standard Methods** are mentioned in Table 10..2 and their details are discussed in the subsequent sections.

Table 10.2: Testing procedures/methods as per **Standard Methods**[16]

Sr. No.	Test	Procedure as per Standard Methods.
1.	pH	pH meter with two point calibration
2.	TSS	APHA-2540 D
3.	BOD	ASTM-5210 B
4.	COD	APHA-5220 D

10.4.1 pH

The concept of pH was first introduced by DanishchemistS. P. L. Sørensen at the Carlsberg Laboratory in 1909.

[14]Standard Methods for the Examination of Water and Wastewater, American Public Health Association, American Water Works Association and Water and Environment Federation, USA.

pH is a measure of the acidity or alkalinity of a solution. Aqueous solutions at 25°C with a pH less than seven are considered acidic, while those with a pH greater than seven are considered basic (alkaline). When a pH level is 7.0, it is defined as 'neutral' at 25°C because at this pH the concentration of $H3O^+$ equals the concentration of OH^- in pure water. pH is formally dependent upon the activity of hydroniumions (H3O+) but for very dilute solutions, the molarity of H3O+ may be used as a substitute with little loss of accuracy. (H+ is often used as a synonym for H3O+.) Because pH is dependent on ionic activity, a property which cannot be measured easily or fully predicted theoretically, it is difficult to determine an accurate value for the pH of a solution. The pH reading of a solution is usually obtained by comparing unknown solutions to those of known pH, and there are several ways to do so.

EQUATION:

$$\mathrm{pH} = -\log_{10} \alpha_{\mathrm{H}^+}$$

Where αH+ denotes the activity of H+ ions, and is dimensionless. In solutions containing other ions, activity and concentration will not generally be the same. Activity is a measure of the effective concentration of hydrogen ions, rather than the actual concentration; it includes the fact that other ions surrounding hydrogen ions will shield them and affect their ability to participate in chemical reactions. These other ions change the effective amount of hydrogen ion concentration in any process that involves H+.

PH OF PURE WATER:

In solution at 25 °C, a pH of 7 indicates neutrality (i.e. the pH of pure water) because water naturally dissociates into H+ and OH− ions with equal concentrations of 1×10−7 mol/L. A lower pH value (for example pH 3) indicates increasing strength of acidity, and a higher pH value (for example pH 11) indicates increasing strength of basicity. Note, however, that pure water, when exposed to the atmosphere, will take in carbon dioxide, some of which reacts with water to form carbonic acid and H+, thereby lowering the pH to about 5.7.The pH of water gets smaller with higher temperatures. For example, at 50 °C, pH of water is 6.55 ± 0.01. This means that a diluted solution is neutral at 50 °C when its pH is around 6.55 and that a pH of 7.00 is basic.

PH ADJUSTMENT:

Distilled water has an average pH of 7 (neither alkaline nor acidic) and sea water has an average pH of 8.3 (slightly alkaline). If the water is acidic (lower than 7), lime or soda ash is added to raise the pH. Lime is the more common of the two additives because it is cheap, but it also adds to the resulting water hardness.

PH MEASUREMENT:

PH can be measured by addition of a pH indicator into the solution under study. The indicator color varies depending on the pH of the solution.

Representative pH values	
Substance	pH
Hydrochloric acid, 10M	-1.0
Lead-acid battery	0.5
Gastric acid	1.5 – 2.0
Lemon juice	2.4
Cola	2.5
Vinegar	2.9
Orange or apple juice	3.5
Tomato Juice	4.0
Beer	4.5
Acid Rain	<5.0
Coffee	5.0
Tea or healthy skin	5.5
Urine	6.0
Milk	6.5
Pure Water	7.0
Healthy humansaliva	6.5 – 7.4
Blood	7.34 – 7.45
Seawater	7.7 – 8.3
Hand soap	9.0 – 10.0
Household ammonia	11.5
Bleach	12.5
Household lye	13.5

PROCEDURE:

1- Caliberate the Instrument

a- After using of apparatus for various liquids standard solution pH change so in order to perform next experiment we have to make standard solution p^H within neutral range

b- So for this purpose BUFFER SOLUTIONS are used to neutralize the standard solution used for testing because buffer solutions resist against change in p^H of the solution

c- BUFFER SOLUTIONS are of two types which are as follows

i- Acidic Buffer

Acidic buffer solution is combination of strong base and salt
EXAMPLE
CH_3COOH and CH_3COONa

ii- Basic Buffer

Basic buffer solution is combination of weak base and salt

2- Switch On the Instrument

3- Take that sample whose pH is to be determined

4- Insert glass electrode probe of the p^H meter in the solution which was initially present in standard buffer solution and wait for 5-10 minutes when blinking quotation of stabilizing on the screen of p^H meter stops note that reading

5- Take the glass electrode probe out form the solution and placed again in standard Buffer solution

6- For next experiment repeat the previous procedure

OBSERVATION AND CALCULATIONS:

SAMPLE NAME	pH OF THE SAMPLE	TEMPERATURE °C

COMMENTS: by lab staff, if any

10.4.2 TOTAL SUSPENDED SOLIDS (TSS)

Definition

It refers to the solid material in water or wastewater sample, which is retained by a filter of 2.0 µm (or smaller) nominal size under specified conditions.

Purpose

TSS is an important parameter in wastewater treatment. The data obtained from this test is used in the design of wastewater treatment plants. In addition to this, TSS is a parameter mentioned in NEQS.

Scope

This procedure is applicable to waters of wide range of quality including surface water, industrial and domestic effluents, treated wastewaters etc.

Interferences

1. Sampling, sub-sampling, and pipeting two-phase samples may introduce serious errors. Homogenization of sample before sampling and use of wide-mouth pipette eliminates the error;
2. Exclude large floating particles or submerged agglomerates of non-homogeneous materials from the sample if it is determined that their inclusion is not representative;
3. Excessive solids may form water-trapping crust, which can be avoided by limiting the sample to produce 0.2 g residue;
4. For samples with high TDS thoroughly wash the filter to ensure removal of dissolved material;
5. To aid in the quality assurance, analyze samples in duplicate.

Sampling and Storage

1. Obtain an appropriate quantity of sample in a glass or plastic bottle;
2. If suspended matter adheres to the container walls prefer glass container;
3. Analyze sample as soon as possible. If preservation required, store it at 4 ^{0}C to avoid biological degradation;
4. Preferably analyze with 24 hours and in no case after 7 days.
5. Bring sample to room temperature before analysis.

Apparatus

1. Muffle furnace for operating at 550^{0}C;
2. Desiccator provided with a desiccant containing a color indicator of moisture concentration;
3. Drying oven, for operation at 103 to 105 ^{0}C;
4. Analytical balance capable of weighing 0.1 mg;
5. Magnetic stirrer.
6. Wide-mouth pipettes;
7. Glass fiber filter disks without organic binder;

8. Filtration apparatus;
9. Suction flasks sufficient capacity for sample size selected;
10. Aluminum weighing dishes.

Procedure

1. Preparation of glass fiber filter disk: Insert disk with wrinkled side up into filtration apparatus;
2. Apply vacuum and wash disk with three successive 20 mL volumes of distilled water. Continue suction to remove all traces of water and discard washings;
3. Remove filter from filtration apparatus and transfer to an inert aluminum weighing dish;
4. Dry the glass fiber filter disk in an oven at 103 to105 ^{0}C for 1 hour. If volatile solids are to be measured, ignite at 550 ^{0}C for 15 minutes in a muffle furnace;
5. Cool in a desiccator to balance temperature and weigh;
6. Repeat cycle of drying or igniting, cooling, desiccating, and weighing until a constant weight is obtained or until weight change is less than 4% of the previous weighing or 0.5 mg, whichever is less. Store in a desiccator until needed;
7. Choose sample volume to yield between 2.5 and 200 mg dried residue. If volume filtered fails to meet minimum yield, increase the sample volume up to 1 L. If more than 10 minutes are required for complete filtration, increase the filter size or decrease sample volume;
8. Assemble filtering apparatus and filter and begin suction. Wet filter with a small volume of distilled water to seat it;
9. Pipette a measured volume of well-mixed sample onto the seated glass fiber filter disk (pre-dried and weighed) with applied vacuum;
10. Wash with three successive 10 mL volumes of distilled water, allowing complete drainage between washings, and continue suction for about 3 minutes after filtration is complete. Samples with high TDS may require additional washings;
11. Carefully remove filter from filtration apparatus and transfer to an aluminum weighing dish;
12. Dry for at least 1 hour at 103 to105 °C in an oven, cool in a desiccator to balance temperature and weigh;
13. Repeat drying cycle of drying, cooling, desiccating, and weighing until a constant weight is obtained or until weight loss is less than 4% of previous weight or 0.5 mg, whichever is less.

Calculation

$$\text{TSS, mg/L} = \frac{(A - B) \times 1000}{\text{Sample volume, mL}}$$

Where:

A = weight of dried residue + filter, gm

B = weight of filter, gm

10.4.3 BOD

THEORY

Biochemical Oxygen Demand (BOD)

The amount of oxygen required by the bacteria while stabilizing decomposable organic matter under aerobic conditions. Decomposable means that organic matter can serve as food for the bacteria and energy is derived from its oxidation.

- Biochemical oxygen demand is a measure of the quantity of oxygen used by microorganisms (e.g., aerobic bacteria) in the oxidation of organic matter.
- Natural sources of organic matter include plant decay and leaf fall. However, plant growth and decay may be unnaturally accelerated when nutrients and sunlight are overly abundant due to human influence.
- Urban runoff carries pet wastes from streets and sidewalks; nutrients from lawn fertilizers; leaves, grass clippings, and paper from residential areas, which increase oxygen demand.
- Oxygen consumed in the decomposition process robs other aquatic organisms of the oxygen they need to live. Organisms that are more tolerant of lower dissolved oxygen levels may replace a diversity of more sensitive organisms.

BOD Level (in ppm)	Water Quality
1 - 2	Very Good-not much organic waste present
3 - 5	Moderately clean
6 - 9	Somewhat polluted
10+	Very polluted

Importance of BOD Test in Environmental Engineering

The BOD test is used to determine the relative oxygen requirements of wastewaters, effluents, and polluted waters. The test measures the oxygen utilized during a specified incubation period for the biochemical degradation of organic material. It is also used to determine treatment plant efficiency.

Determination of BOD

Principle:

The method consists of filling with sample, to overflowing, an airtight bottle of the specified size and incubating it at the specified temperature for 5 days. Dissolved oxygen is measured initially and after incubation, and the BOD is computed from the difference between initial and final DO. Because the initial DO is determined shortly after the dilution is made, all oxygen uptake occurring after this measurement is included in the BOD measurement.

Sampling and Storage:

Sample for BOD analysis may degrade significantly during storage between collection and analysis, resulting in low BOD values. Minimize reduction of BOD by analyzing sample promptly or by cooling it to near-freezing temperature during storage. However, even at low temperature, keep holding time to a minimum. Warm chilled samples to 20 ± 3°C before analysis.

Apparatus:

a. Incubation bottles: Use glass bottles having 60 mL or greater capacity (300mL bottles having ground-glass stopper and a flared mouth are preferred).
b. Air incubator or water bath, thermo-statistically controlled at 20 ± 1°C. Exclude all light to prevent possibility of photosynthetic production of DO.

Reagents:

Prepare reagents in advance but discard if there is any sign of precipitation or biological growth in the stock bottles.

a. Phosphate buffer solution: Dissolve 8.5 g KH_2PO_4, 21.75 g K_2HPO_4, 33.4 g $Na_2HPO_4.7H_2O$, and 1.7 g NH_4Cl in about 500 mL distilled water and dilute to 1 Lit. The pH should be 7.2 without further adjustment. Alternatively, dissolve 42.5 g KH_2PO_4or 54.3 g K_2HPO_4in about 700 mL distilled water. Adjust pH to 7.2 with 30% NaOH and dilute to I Lit.

b. Magnesium sulfate solution: Dissolve 22.5 g $MgS0_4.7H_20$ in distilled water and dilute to 1 L.

c. Calcium chloride solution: Dissolve 27.5 $CaCl_2$ in distilled water and dilute to 1 L.

d. Ferric Chloride solution: Dissolve 0.25 g $FeCl_3.6H_2O$ in distilled water and dilute to 1 L.

e. Acid and alkali solution, 1N, for neutralization of caustic or acidic waste samples. 1) Acid-Slowly and while stirring, add 28 mL cone. Sulfuric acid to distilled Water. Dilute to 1 L. 2) Alkali-Dissolve 40 g sodium hydroxide in distilled water. Dilute to 1 L.

f. Sodium sulfate solution: Dissolve 1.575 g Na_2SO_3 in 1000 mL distilled water. This solution is not stable; prepare daily.

g. Nitrification inhibitor: 2-chloro-6-(trichloromethyl) pyridine (if nitrification inhibition desired).

h. Glucose-glutamic acid solution: Dry reagent-grade glucose and reagent-grade glutamic acid at 103°C for 1 h. Add 150 mg glucose and 150 mg glutamic acid to distilled water and dilute to 1 L. Prepare fresh immediately before use.

i. Ammonium chloride solution: Dissolve 1.15 g NH_4Cl in about 500 mL distilled water, adjust pH to 7.2 with NaOH solution and dilute to 1 L. Solution contains 0.3 mg N mL^{-1}

j. Dilution water: Use demineralized, distilled, tap, or natural water for making sample dilutions.

Procedure:

1) First of all it is important to know the amount of samples to be used for test. For this purpose the source of sample is to be recorded which will indicate the approximate value of BOD_5 for the sample.
   (i) Domestic sewage BOD_5 =100-500mg/L
   (ii) Effluent from treatment plant= 20-80mg/L
   (iii) River water = 2-4mg/L
2) Take 9 BOD bottles note their numbers and arrange them in 3 groups.

3) Fill each bottle half with dilution media ensuring that no air gets mixed with the media while fill in as in DO test.
4) Add 2ml sample in each of the three bottles marked as first group; 5 ml in each bottle of 2nd group and 10ml in each bottle of the 3rd group.
5) Fill the bottle completely with dilution media and place the stopper such that no air bubbles are trapped.
6) Now take one bottle from each set and estimate its DO. This will be DO initial or DO 0days.
7) For comparison prepare two more bottles with blank dilutions media (with out sewage sample) and find the DO from one bottle.
8) Place the rest of the six bottles with sewage samples and one bottle for blank in the incubator at 20^0 C.
9) After 5 days find out DO in all bottles.
10) That value of oxygen depletion should be considered correct which gives an oxygen depletion of at least 2 mg/L. and which have at least 0.5 mg/L DO after 5 days of incubation.
11) Calculate BOD at 20^0 C. for the sample using following relationship.

$$\text{BOD(mg/L)} = \frac{\text{DO depletion (mg/L)} * 300}{\text{Volume of sample in bottle (ml)}}$$

Observations:

<u>At zero days.</u>

Bottle#	Sample added (ml)	Volume of sample (ml)	Volume of $Na_2S_2O_3$	DO (mg/L)
	Blank			

After 5 days.

Bottle#	Sample added (ml)	Volume of sample (ml)	Volume of $Na_2S_2O_3$	DO (mg/L)
	Blank			

DO Depletion:

Bottle#	Sample added (ml)	DO at Zero days (mg/L)	DO at 5 days (mg/L)	DO Depleted (mg/L)	BOD (mg/L)
	Blank				

Mean BOD =

<u>Comments:</u> by lab staff, if any

10.4.4 COD

THEORY

Chemical Oxygen Demand (COD)

The Chemical Oxygen Demand, or COD,is a measurement of the amount of material that can be oxidized (combined with oxygen) in the presence of a strong chemical oxidizing agent. Since the COD test can be performed rapidly, it is often used as a rough approximation of the water's BOD, even though the COD test measures some additional organic matter (such as cellulose) which is not normally oxidized by biological action. As with the BOD test, the COD test is reported as mg/Lit of oxygen used. The table below shows the normal range of COD found in various kinds of domestic wastewater. Keep in mind that the addition of industrial waste can cause these values to vary widely. Biochemical oxygen demand is a measure of the quantity of oxygen used by microorganisms (e.g. aerobic bacteria) in the oxidation of organic matter.

METHODS OF DETERMINATION OF COD

1. Open Reflux Titrimetric Method

Principle

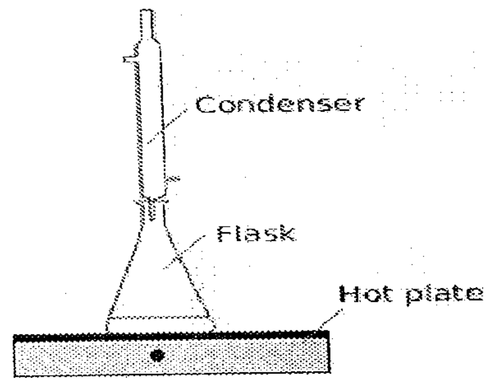
In this method known amount of strong oxidizing agent is being added. Then reaction takes place to form CO_2 and H_2O. Then remaining amount of oxidizing agent is being determined by titration. The amount of oxidizing agent to be added depends upon the COD of sample which can roughly be known by knowing the source of sample.

Equipment:

Caution: The presence of minute traces of organic matter on the equipment will cause large errors in the test results. So clean all equipment thoroughly before using.

1. Erlenmeyer flask
2. Small beaker
3. Titration apparatus:
    a. 25 or 50 mL burette, graduated in 0.1 mL
    b. burette support
    c. 100 mL graduated cylinder
    d. rubber-tipped stirring rod, or magnetic stirrer and stir bar
    e. white porcelain evaporating dish, 4.5 inches in diameter

4. Reflux apparatus:



5. 500 or 250 mL Erlenmeyer flasks with ground glass 24/40 neck
6. 300 mm jacket or equivalent condenser with 24/40 ground-glass joint
7. hot plate with sufficient power to produce at least 1.4 W /cm^2 of heating surface
8. Blender
9. Pipets
10. Glass beads
11. Fume hood

Reagents

1. Standard potassium dichromate solution, 0.25N or O.025N
2. Sulfuric acid reagent containing silver sulfate catalyst
3. Standard ferrous ammonium sulfate titrant
4. Ferroin indicator solution
5. Mercuric sulfate crystals
6. Sulfuric acid
7. Concentrated sulfuric acid
8. Distilled water

Theory of Titration

The COD analysis, by the dichromate method, is more commonly used to control and continuously monitor wastewater treatment systems, The COD of an effluent is usually higher than the BOD_5 since the number of compounds that can be chemically oxidized is greater than those that can be degraded biologically, It is also common to make a correlation of BOD_5 versus COD and then use the analysis of COD as a rapid means of estimating the BOD_5 of a wastewater. This may be convenient since only about three hours are needed for a COD determination while a BOD_5 takes at least 5 days. However, this procedure can be used only for specific situations where there is low variability in the composition of a wastewater, and the results of a system cannot be used reliably in other cases.

The method of COD which uses dichromate as oxidant is carried out by heating under total reflux a wastewater sample of known volume in an excess of potassium dichromate ($K_2Cr_2O_7$) in presence of sulphuric Acid (H_2SO_4) for a fixed period (usually two hours) in presence of silver sulphate (Ag_2SO_4) as catalyst. The organic matter present is oxidized and, as a result, the dichromate ion (orange colour) is consumed and replaced by the chromic ion (green colour):

$$\mathbf{Cr_2O_7^{-2} + 14H^+ + 6e^- \longrightarrow 2Cr^{3+} + 7H_2O}$$

The COD is calculated by titrating the excess of dichromate or by spectrophotometrically measuring the Cr^{+3} ions at 606 nm. Another possibility is to measure the excess dichromate spectrophotometrically at 440 nm. Titration requires more work but is considered more precise.

The presence of silver sulphate as catalyst is needed for complete oxidation of aliphatic carbon compounds. The standard method implies cooling of the sample after the two hour digestion period, adding a few drops of indicator (ferroin) solution and titrating the excess dichromate with a solution of ferrous ammonium sulphate of known concentration, until the colour changes from brilliant green to reddish brown. The titration reaction corresponds to the oxidation of the ferrous ammonium sulphate by the dichromate:

$$\mathbf{Cr_2O_7^{-2} + 14H^+ + 6Fe^{+2} \longrightarrow 2Cr^{3+} + 6Fe^{+3} + 7H_2O}$$

The change in colour corresponds to the formation of the complex ferrous ion phenanthroline which occurs when all the dichromate ion has been reduced to Cr^{3+}.

$$\mathbf{(Fe(C_{12}H_8N_2)_3)^{3+} + e \longrightarrow (Fe(C_{12}H_8N_2)_3)^{2+}}$$

Ferric Phenanthroline	Ferrous Phenanthroline
(Green Blue)	(Reddish Brown)

Interferences

A common interference factor in the COD test is the presence of chlorides. If seawater is used at some point in the processing or salt brines are used for some "curing" operations, chlorides will most probably appear in the wastewater causing interference while they are oxidized by the dichromate:

$$\mathbf{Cl^- + Cr_2O_7^{2-} + 14H^+ \longrightarrow 3Cl_2 + 2Cr^{3+} + 7H_2O}$$

This interference causes erroneously high values of COD which can be prevented by the addition of mercuric sulphate ($HgSO_4$) which reacts to form mercuric chloride and precipitates:

$$Hg^{2+} + 2Cl^{-} \longrightarrow HgCl_2$$

Procedure:

1) Place 50ml sample in 500ml refluxing flask (for samples with COD>900mg/L use a smaller sample diluted to 50ml).
2) Add 1g $HgSO_4$ and several glass beeds.
3) Add slowly 5ml H_2SO_4 reagent while mixing to dissolve $HgSO_4$
4) Cool while mixing to avoid the loss of volatile materials.
5) Add 25 ml 0.25N $K_2\ Cr_2O_7$ solution and mix.
6) Attach the flask to the condenser and turn on cooling water.
7) Add remaing H_2SO_4 (70mL) through open end of the condenser continue mixing while adding H_2SO_4.
8) Reflux the mixture for 2 hrs and cool to room temperature, after diluting the mixture to about twice its volume with distilled water.
9) Titrate excess of $K_2\ Cr_2O_7$ with Ferrous ammonium sulfate using 2,3 drops of ferrion indicator. The end point will be from blue green to reddish brown.
10) Reflux and titrate in the same manner a blank containing the reagents and the voume of the distilled water will be equal to that of sample.

<u>OBSERVATIONS AND CALCULATIONS (COD)</u>

Sr. No.	Description of Sample	Volume of titrant used for sample	Volume of titrant used for blank	COD
		ml	ml	mg/L
1				
2				

COD, mg/L = (A - B) x N x 8,000 / (Volume of Sample, mL)

Where:

A = mL of titrant used for Blank

B = mL of titrant used for Sample

N = normality of ferrous ammonium sulfate (FAS) = 0.25N

8000 = Equivalent Wt. of Oxygen x 1000

<u>Comments:</u> by lab staff, if any.

Chapter- 11

DESIGN CALCULATION FOR ONE COMBINED WWTP FOR GUJRANWALA

11.1 DESIGN DATA

Design population (2038) = 2,628,016 persons

Per capital water consumption = 50 GPCD (225 LPCD)

Wastewater flow = 80% of water consumption

Per capita wastewater flow = 50x0.8= 40 GPCD (180 LPCD)

BOD of wastewater = 250 mg/L

Design wastewater flow = 180x2, 628,016=473,042 m^3/day (193 cusec)

11.2 DESIGN CALCULATION

Following design calculations are for one combined WWTP based on WSP for the entire Gujranwala city. The WSP has been divided into multiple modules. Each module consists of one AP and one FP, in series. The benefit of having multiple modules is that the construction activity can be undertaken in phases. New modules can be added to meet future demands as the sewage flow increases with the passage of time. However, the area for entire WWTP should be acquired in the beginning. Different flows for one module may be adopted.

For Gujranwala, flow of one module adopted= 12,000 m^3/day

Size calculation for one module has been performed below. Total number of modules for one combined WWTP has also been evaluated.

Parameter			Unit	Quantity
Flow of one module	Q		m^3/day	12,000
Population served by one module	P		persons	66,667
Influent BOD	Li		mg/L	250
Design Temperature (mean temp of coldest month)	T		°C	16
ANAEROBIC POND				
Design Criteria				
The design of anaerobic ponds is based on the following parameters:				
Detention time			days	1 - 5
Solid accumulation rate (SAR)			m^3/person/yr	0.03 - 0.04
Depth			m	3 - 4.5
Allowable BOD loading λv=(20T-100) when T=10-20°C				
Allowable BOD loading λv=(10T+100) when T=20-25°C				
BOD removal in Anaerobic pond (2T+20)				
(**Reference:** Duncan Mara (1997), Design Manual For Waste Stabilization Ponds)				
Design Calculation				
Allowable BOD loading λv=(20T-100) when T=10-20°C			g/m^3.day	220
Volume of anaerobic pond: $\lambda_v = \frac{L_i Q}{V}$	V		m^3	13,636
Actual detention Time	Dt	Va/Q	days	1.14
Adopted detention time for better efficiency	Dta		days	2

Parameter			Unit	Quantity
Adopted volume of anaerobic pond	Va		m^3	24,000
Sludge Accumulation rates (SAR)			m^3/person/yr	0.03
Volume of sludge accumulation per year	V1	P x SAR	m^3	2,000
Let Frequency for desludging @ 3 year	V3	V1 x 3	m^3	6,000
Total Volume (Vt)	Vt	V3+ Va	m^3	30,000
Let wastewater depth in Anaerobic Ponds		D	m	4.0
Free Board			m	1.0
Total Depth			m	5.0
Mid depth area of Anaerobic ponds	Am=	Vt/D	m^2	7,500
Take side slopes as 1 Vertical : 3 Horizontal				
Removal efficiency= 2T+20			%	52.00
Effluent BOD from anaerobic pond			mg/L	120.00
Mid Depth Area of Pond	A1	V/d1	m^2	7,500
Adopting Water Depth of pond	d1		m	4.00
Free Board	d1f		m	1.00
Total Depth			m	5.00
L:W Ratio			r	2.00
Mid Depth Length	Lm	$\sqrt{(Am/r)}$	m	61.24

Parameter			Unit	Quantity
Mid Depth Width	Wm	Lm x r	m	122.47
Top Length	Lt	Lm +18	m	79.24
Top Width	Wt	Wm +18	m	140.47
Bottom Length	Lb	Lm -12	m	49.24
Bottom Width	Wb	Wm - 12	m	110.47
Top Area of Anaerobic Pond	At	LtxWt	m^2	11,131
	At		Hectare	1.11
BOD removal in Anaerobic Pond (2T+20)		%	52	
BOD after treatment in Anaerobic pond			mg/L	120.0
FACULTATIVE POND				
Design Criteria				
The design of Facultative ponds is based on the following parameters:				
Detention time			days	5-12
Depth			m	2 - 2.5
Side slopes			Vert:Horiz	1:3
Allowable BOD loading λs=(20T-60) when T=10-20°C			Kg/hectare.day	
BOD removal in Facultative pond is given by following formula				
Effluent BOD		Le=	$\frac{L_i}{(1+Kt)}$	

Parameter			Unit	Quantity
Where				
K = BOD removal rate constant			per day	0.2
t = Detention time			days	
Design Calculation				
Design wastewater flow	Q		m^3/day	12,000
Allowable Surface loading rate λs = (20T-60)	λs		kg/ha.day	260
Influent BOD for Facultative ponds (Li)	Li		mg/L	120.0
Mid depth area of Facultative pond Af = (10LiQ)/ λs	Af		m^2	55,385
Take wastewater depth of faculatative pond	Df		m	2.00
Free Board			m	1.00
Total Depth			m	3.00
Volume of facultative pond	Vf	AfxDf	m^3	110,769
Actual detention time in facultative pond	Dt	Vf/Q	days	9.23
Top Width	W2t	Wt	m	140
Mid Depth Width	W2m	W2t-12	m	128
Mid depth length	L2m	Af/W2m	m	431
Top length	L2t	L2m + 12	m	443
Bottom width	W2b	L2m - 6	m	122
Bottom length	L2b	L2m-6	m^2	425
Total area of Facultative pond	WtxLt		m^2	62,243
			Hectare	6.2

Parameter			Unit	Quantity
BOD removal in facultative ponds				
BOD of the finally treated wastewater	Le	=	$\frac{L_i}{(1+Kt)}$	
	Le	=	mg/L	42
Effluent BOD Standard in Pakistan		**=**	**mg/L**	**80**
Total area of Anaerobic and Facultative ponds of one module		=	Hectare	7.3
Add 50 % more for bunds, buffer zone and services		=	Hectare	3.67
Total area required for ONE MODULE of WSP system		=	Hectare	11.01
Design discharge (2038)	**Qd**		**m^3/day**	**473,042**
No of modules required for WWTP	Qd/Q			39.4
			say	40.0
DESIGN SUMMARY				
Effluent BOD		=	mg/L	42
Top Length of Anaerobic Pond		=	m	79.0
Top Width of Anaerobic Pond		=	m	140.0
Total Depth of Anaerobic Pond		=	m	5.00
Top Width of Facultative Pond		=	m	140.0
Top Length of Facultative Pond		=	m	443.0
Total Depth of Facultative Pond		=	m	3.00

11.3 LAYOUT OF COMBINED WWTP

The layout of combined WWTP, typical section of one module of WSP, plan and elevation of other facilities are given in the drawings attached at the end of this feasibility report.

11.4 APPROXIMATE COST OF ONE COMBINED WWTP

The approximate cost of combined WWTP having different components has been shown in Table 11.1

Table 11.1: Approximate cost of combined WWTP for Gujranwala city

Sr. No.	Description	Unit	Qty.	Rate (Rs.)	Amount (Rs.)	Amount (in Million Rs.)
1	**Waste Stabilization Ponds**					
	Cost of 1 module					
I	Excavation	Cft	3,788,210	1.76	6,660,502	
Ii	Filling	Cft	756,551	1.37	1,034,586	
iii	Internal Lining	Sft	180,049	105.25	18,949,868	
iv	Outer Lining	Sft	71,638	105.25	7,539,856	
v	Bed Area	Sft	619,014	105.25	65,150,516	
vi	Brick lining at Top	Sft	96,977	105.25	5,405,124	
vii	Railing	Rft	4,351	55.74	1,296,828	
viii	Toe Length	Rft	4,641	298.08	2,121,828	
Ix	Drains	Rft	6562	1905	12,500,000	
					120,659,000	
	Total Cost of 40 Modules				**4,826,368,000**	**4,826.37**
2	**Cost of Land**	Acre	1,062	500,000	**531,000,000**	**531**
3	**Site External Works**					
I	Approach Road	1000	5.74	915,000	5,250,000	

Sr. No.	Description	Unit	Qty.	Rate (Rs.)	Amount (Rs.)	Amount (in Million Rs.)
		Rft				
ii	Track	1000 Rft	5.20	1,000,000	5,200,000	
iii	Fencing	Rft	28,000	153	4,284,000	
iv	Parking Area	Sft	4,000	85	340,000	
v	Sewerage	Lumpsum			5,000,000	
vi	Electrical	Lumpsum			2,697,800	
vii	Piping Cost	Rft	5,000	800	4,000,000	
viii	Gate	No.	2	120,000	240,000	
	Total Cost of Site External Works				**27,012,000**	**27.01**
4	**Office Block**	Sft	2,336	1,580	**3,690,000**	**3.69**
5	**Staff Quarters**	Sft	700	1,395	**976,000**	**0.98**
6	**Laboratory**	Sft	2,336	1,395	**1,711,952**	**1.71**
7	**Lab. Equipment**	Lumpsum			**1,100,000**	**1.10**
8	**Store / Shed**	Sft	2,336	1,395	**1,637,000**	**1.64**
9	**Sewage pumping station**					
i	Wet well Cost	Sft	24,500	1,200	29,400,000	
ii	Machinery Cost				480,000,000	
	Total Cost of Sewage Pumping Station				**509,400,000**	**509.40**
10	**Miscellaneous**	Lumpsum			**10,000,000**	**10**
	Total Cost				**5,912,894,000**	**5912.90**

Chapter- 12

DESIGN CALCULATION FOR MULTIPLE WWTP FOR GUJRANWALA

12.1 GENERAL

As discussed in Chapter-5, there are two options available for undertaking the wastewater treatment in Gujranwala city. One option is to have a combined WWTP for the entire city. Second option is to have multiple WWTPs. The second option is better with respect to the reasons stated in chapter-5. This chapter deals with the design calculation for the multiple WWTPs.

12.2 WWTP QILA MIAN SINGH

Design discharge = 156,140 m^3/day (64 cusec)

Per capita sewage flow = 50 GPCD x 4.5 Liter x 80% ÷1000=0.18 m^3/day

Design Population = 156,140/0.18=867,444 persons

Design Calculations

Flow for one module = 12000 m^3/day

Parameter		Unit	Quantity
Flow of one module	Q	m^3/day	12,000
Population served by one module	P	persons	66,667
Influent BOD	Li	mg/L	250
Design Temperature (mean temp of coldest month)	T	°C	16
ANAEROBIC POND			
Design Criteria			
The design of anaerobic ponds is based on the following parameters:			
Detention time		days	1 - 5

Parameter			Unit	Quantity
Solid accumulation rate (SAR)			m^3/person/yr	0.03 - 0.04
Depth			m	3 - 4.5
Allowable BOD loading λv=(20T-100) when T=10-20°C				
Allowable BOD loading λv=(10T+100) when T=20-25°C				
BOD removal in Anaerobic pond (2T+20)				
(**Reference:** Duncan Mara (1997), Design Manual For Waste Stabilization Ponds)				
<u>Design Calculation</u>				
Allowable BOD loading λv=(20T-100) when T=10-20°C			g/m^3.day	220
Volume of anaerobic pond: $\lambda_v = \frac{L_i Q}{V}$	V		m^3	13,636
Actual detention Time	Dt	Va/Q	days	1.14
Adopted detention time for better efficiency	Dta		days	2
Adopted volume of anaerobic pond	Va		m^3	24,000
Sludge Accumulation rates (SAR)			m^3/person/yr	0.03
Volume of sludge accumulation per year	V1	P x SAR	m^3	2,000
Let Frequency for desludging @ 3 year	V3	V1 x 3	m^3	6,000
Total Volume (Vt)	Vt	V3+ Va	m^3	30,000
Let wastewater depth in Anaerobic Ponds		D	m	4.0
Free Board			m	1.0

Parameter			Unit	Quantity
Total Depth			m	5.0
Mid depth area of Anaerobic ponds	Am=	Vt/D	m^2	7,500
Take side slopes as 1 Vertical : 3 Horizontal				
Removal efficiency= 2T+20			%	52.00
Effluent BOD from anaerobic pond			mg/L	120.00
Mid Depth Area of Pond	A1	V/d1	m^2	7,500
Adopting Water Depth of pond	d1		m	4.00
Free Board	d1f		m	1.00
Total Depth			m	5.00
L:W Ratio			r	2.00
Mid Depth Length	Lm	√(Am/r)	m	61.24
Mid Depth Width	Wm	Lm x r	m	122.47
Top Length	Lt	Lm +18	m	79.24
Top Width	Wt	Wm +18	m	140.47
Bottom Length	Lb	Lm -12	m	49.24
Bottom Width	Wb	Wm - 12	m	110.47
Top Area of Anaerobic Pond	At	LtxWt	m^2	11,131
	At		Hectare	1.11

Parameter	Unit	Quantity
BOD removal in Anaerobic Pond (2T+20) %	52	
BOD after treatment in Anaerobic pond	mg/L	120.0
FACULTATIVE POND		
Design Criteria		
The design of Facultative ponds is based on the following parameters:		
Detention time	days	5-12
Depth	m	2 - 2.5
Side slopes	Vert:Horiz	1:3
Allowable BOD loading λs=(20T-60) when T=10-20°C	Kg/hectare.day	
BOD removal in Facultative pond is given by following formula		
Effluent BOD Le=	$\frac{L_i}{(1 + Kt)}$	
Where		
K = BOD removal rate constant	per day	0.2
t = Detention time	days	
Design Calculation		
Design wastewater flow Q	m^3/day	12,000
Allowable Surface loading rate λs = (20T-60) λs	kg/ha.day	260
Influent BOD for Facultative ponds (Li) Li	mg/L	120.0

Parameter			Unit	Quantity
Mid depth area of Facultative pond Af = (10LiQ)/ λs	Af		m^2	55,385
Take wastewater depth of faculatative pond	Df		m	2.00
Free Board			m	1.00
Total Depth			m	3.00
Volume of facultative pond	Vf	AfxDf	m^3	110,769
Actual detention time in facultative pond	Dt	Vf/Q	days	9.23
Top Width	W2t	Wt	m	140
Mid Depth Width	W2m	W2t-12	m	128
Mid depth length	L2m	Af/W2m	m	431
Top length	L2t	L2m + 12	m	443
Bottom width	W2b	L2m - 6	m	122
Bottom length	L2b	L2m-6	m^2	425
Total area of Facultative pond	WtxLt		m^2	62,243
			Hectare	6.2
BOD removal in facultative ponds				
BOD of the finally treated wastewater	Le	=	$\frac{L_t}{(1+Kt)}$	
	Le	=	mg/L	42
Effluent BOD Standard in Pakistan		**=**	**mg/L**	**80**
Total area of Anaerobic and Facultative ponds of one module		=	Hectare	7.3

Parameter		Unit	Quantity
Add 50 % more for bunds, buffer zone and services	=	Hectare	3.67
Total area required for ONE MODULE of WSP system	=	Hectare	11.01
Design discharge (2038)	**Qd**	**m^3/day**	156,140
No of modules required for WWTP	Qd/Q		13.0
		say	14.0
DESIGN SUMMARY			
Effluent BOD	=	mg/L	42
Top Length of Anaerobic Pond	=	m	79.0
Top Width of Anaerobic Pond	=	m	140.0
Total Depth of Anaerobic Pond	=	m	5.00
Top Width of Facultative Pond	=	m	140.0
Top Length of Facultative Pond	=	m	443.0
Total Depth of Facultative Pond	=	m	3.00

12.2.1 Drawing for WWTP QilaMian Singh

The drawing are attached at the end of this report.

12.2.2 Approximate Cost of WWTP QilaMian Singh

The approximate cost is shown in Table 12.1

The approximate cost of WWTP QilaMian Singh having different components has been shown in Table 12.1

Table 12.1: Approximate cost of WWTP QilaMian Singh

Sr. No.	Description	Unit	Qty.	Rate (Rs.)	Amount (Rs.)	Amount (in Million Rs.)
1	**Waste Stabilization Ponds**					
	Cost of 1 module					
I	Excavation	Cft	3,788,210	1.76	6,660,502	
Ii	Filling	Cft	756,551	1.37	1,034,586	
iii	Internal Lining	Sft	180,049	105.25	18,949,868	
iv	Outer Lining	Sft	71,638	105.25	7,539,856	
v	Bed Area	Sft	619,014	105.25	65,150,516	
vi	Brick lining at Top	Sft	96,977	105.25	5,405,124	
vii	Railing	Rft	4,351	55.74	1,296,828	
viii	Toe Length	Rft	4,641	298.08	2,121,828	
Ix	Drains	Rft	6562	1905	12,500,000	
					120,659,000	
	Total Cost of 14 Modules				**1,689,226,000**	**1,689.23**
2	**Cost of land**	Acre	430	700,000	**301,000,000**	**301.00**
3	**Site External Works**					
I	Approach Road	1000 Rft	4.92	915,000	4,501,800	
ii	Track	1000 Rft	1.26	1,000,000	1,260,000	
iii	Fencing	Rft	17,560	153	2,686,700	
iv	Parking Area	Sft	2,000	85	170,000	

Sr. No.	Description	Unit	Qty.	Rate (Rs.)	Amount (Rs.)	Amount (in Million Rs.)
v	Sewerage	Lumpsum			3,000,000	
vi	Electrical	Lumpsum			2,697,800	
vii	Piping Cost	Rft	2,000	800	1,600,000	
viii	Gate	No.	1	120,000	120,000	
	Total Cost of Site External Works				**16,036,300**	**16.04**
4	**Office Block**	Sft	2,336	1,580	**3,690,000**	**3.69**
5	**Staff Quarters**	Sft	700	1,395	**976,000**	**0.98**
6	**Laboratory**	Sft	2,336	1,395	**1,711,952**	**1.71**
7	**Lab. Equipment**	Lumpsum			**1,100,000**	**1.10**
8	**Store / Shed**	Sft	2,336	1,395	**1,637,000**	**1.64**
9	**Sewage pumping station**					
i	Wet well Cost	Sft	8,750	1,200	10,500,000	
ii	Machinery Cost				156,000,000	
	Total Cost of Sewage Pumping Station				**166,500,000**	**166.50**
10	**Miscellaneous**	Lumpsum			**5,000,000**	**5.00**
	Total Cost				**2,186,877,000**	**2186.88**

12.3 WWTP MAIN DRAIN AND JINNAH DRAIN

Design discharge = 263,989 m^3/day (108 cusec)

Per capital sewage flow = 0.18 m^3/day

Design Population = 263,989/0.18=1,466,605 persons

Flow for one module = 12,000 m^3/day

Design Calculations

Parameter			Unit	Quantity
Flow of one module	Q		m^3/day	12,000
Population served by one module	P		persons	66,667
Influent BOD	Li		mg/L	250
Design Temperature (mean temp of coldest month)	T		°C	16
ANAEROBIC POND				
Design Criteria				
The design of anaerobic ponds is based on the following parameters:				
Detention time			days	1 - 5
Solid accumulation rate (SAR)			m^3/person/yr	0.03 - 0.04
Depth			m	3 - 4.5
Allowable BOD loading λv=(20T-100) when T=10-20°C				
Allowable BOD loading λv=(10T+100) when T=20-25°C				
BOD removal in Anaerobic pond (2T+20)				
(**Reference:** Duncan Mara (1997), Design Manual For Waste Stabilization Ponds)				
Design Calculation				
Allowable BOD loading λv=(20T-100) when T=10-20°C			g/m^3.day	220
Volume of anaerobic pond: $\lambda_v = \frac{L_i Q}{V}$	V		m^3	13,636
Actual detention Time	Dt	Va/Q	days	1.14
Adopted detention time for better efficiency	Dta		days	2

Parameter			Unit	Quantity
Adopted volume of anaerobic pond	Va		m^3	24,000
Sludge Accumulation rates (SAR)			m^3/person/yr	0.03
Volume of sludge accumulation per year	V1	P x SAR	m^3	2,000
Let Frequency for desludging @ 3 year	V3	V1 x 3	m^3	6,000
Total Volume (Vt)	Vt	V3+ Va	m^3	30,000
Let wastewater depth in Anaerobic Ponds		D	m	4.0
Free Board			m	1.0
Total Depth			m	5.0
Mid depth area of Anaerobic ponds	Am=	Vt/D	m^2	7,500
Take side slopes as 1 Vertical : 3 Horizontal				
Removal efficiency= 2T+20			%	52.00
Effluent BOD from anaerobic pond			mg/L	120.00
Mid Depth Area of Pond	A1	V/d1	m^2	7,500
Adopting Water Depth of pond	d1		m	4.00
Free Board	d1f		m	1.00
Total Depth			m	5.00
L:W Ratio			r	2.00
Mid Depth Length	Lm	√(Am/r)	m	61.24

Parameter			Unit	Quantity
Mid Depth Width	Wm	Lm x r	m	122.47
Top Length	Lt	Lm +18	m	79.24
Top Width	Wt	Wm +18	m	140.47
Bottom Length	Lb	Lm -12	m	49.24
Bottom Width	Wb	Wm - 12	m	110.47
Top Area of Anaerobic Pond	At	LtxWt	m^2	11,131
	At		Hectare	1.11
BOD removal in Anaerobic Pond (2T+20)		%	52	
BOD after treatment in Anaerobic pond			mg/L	120.0
FACULTATIVE POND				
Design Criteria				
The design of Facultative ponds is based on the following parameters:				
Detention time			days	5-12
Depth			m	2 - 2.5
Side slopes			Vert:Horiz	1:3
Allowable BOD loading λs=(20T-60) when T=10-20°C			Kg/hectare.day	
BOD removal in Facultative pond is given by following formula				
Effluent BOD		Le=	$\frac{L_i}{(1 + Kt)}$	

Parameter			Unit	Quantity
Where				
K = BOD removal rate constant			per day	0.2
t = Detention time			days	
Design Calculation				
Design wastewater flow	Q		m^3/day	12,000
Allowable Surface loading rate λs = (20T-60)	λs		kg/ha.day	260
Influent BOD for Facultative ponds (Li)	Li		mg/L	120.0
Mid depth area of Facultative pond Af = (10LiQ)/ λs	Af		m^2	55,385
Take wastewater depth of faculatative pond	Df		m	2.00
Free Board			m	1.00
Total Depth			m	3.00
Volume of facultative pond	Vf	AfxDf	m^3	110,769
Actual detention time in facultative pond	Dt	Vf/Q	days	9.23
Top Width	W2t	Wt	m	140
Mid Depth Width	W2m	W2t-12	m	128
Mid depth length	L2m	Af/W2m	m	431
Top length	L2t	L2m + 12	m	443
Bottom width	W2b	L2m - 6	m	122
Bottom length	L2b	L2m-6	m^2	425
Total area of Facultative pond	WtxLt		m^2	62,243
			Hectare	6.2

Parameter			Unit	Quantity
BOD removal in facultative ponds				
BOD of the finally treated wastewater	Le	=	$\frac{L_i}{(1+Kt)}$	
	Le	=	mg/L	42
Effluent BOD Standard in Pakistan		**=**	**mg/L**	**80**
Total area of Anaerobic and Facultative ponds of one module		=	Hectare	7.3
Add 50 % more for bunds, buffer zone and services		=	Hectare	3.67
Total area required for ONE MODULE of WSP system		=	Hectare	11.01
Design discharge (2038)		**Qd**	**m^3/day**	263,989
No of modules required for WWTP		Qd/Q		22.0
			say	22.0
DESIGN SUMMARY				
Effluent BOD		=	mg/L	42
Top Length of Anaerobic Pond		=	m	79.0
Top Width of Anaerobic Pond		=	m	140.0
Total Depth of Anaerobic Pond		=	m	5.00
Top Width of Facultative Pond		=	m	140.0
Top Length of Facultative Pond		=	m	443.0
Total Depth of Facultative Pond		=	m	3.00

12.3.1 Drawing for WWTP MianDrain and Jinnah Drain

The drawings are attached at the end of this report.

12.3.2 Cost of WWTP MianDrain and Jinnah Drain

The approximate cost is shown in Table 12.2

Table 12.2: Rough cost estimate for WWTP Mian Drain and Jinnah Drain

Sr. No.	Description	Unit	Qty.	Rate (Rs.)	Amount (Rs.)	Amount (in Million Rs.)
1	**Waste Stabilization Ponds**					
	Cost of 1 module					
I	Excavation	Cft	3,788,210	1.76	6,660,502	
Ii	Filling	Cft	756,551	1.37	1,034,586	
iii	Internal Lining	Sft	180,049	105.25	18,949,868	
iv	Outer Lining	Sft	71,638	105.25	7,539,856	
v	Bed Area	Sft	619,014	105.25	65,150,516	
vi	Brick lining at Top	Sft	96,977	105.25	5,405,124	
vii	Railing	Rft	4,351	55.74	1,296,828	
viii	Toe Length	Rft	4,641	298.08	2,121,828	
Ix	Drains	Rft	6562	1905	12,500,000	
					120,659,000	
	Total Cost of 22 Modules				**2,654,498,000**	**2,654.50**
2	**Cost of land**	Acre	641	500,000	**302,500,000**	**302.50**
3	**Site External Works**					
I	Approach Road	1000 Rft	1.97	915,000	1,800,720	
Ii	Track	1000	3.950	1,000,000	3,950,000	

Sr. No.	Description	Unit	Qty.	Rate (Rs.)	Amount (Rs.)	Amount (in Million Rs.)
		Rft				
iii	Fencing	Rft	24,052	153	3,680,000	
Iv	Parking Area	Sft	3,000	85	255,000	
V	Sewerage	Lumpsum			4,000,000	
vi	Electrical	Lumpsum			1,881,000	
vii	Piping Cost	Rft	3,000	800	2,400,000	
viii	Gate	No.	1	120,000	120,000	
	Total Cost of Site External Works				**18,087,000**	**18.08**
4	**Office Block**	Sft	2,336	1,580	**3,690,000**	**3.69**
5	**Staff Quarters**	Sft	700	1,395	**976,000**	**0.98**
6	**Store / Shed**	Sft	2,336	1,395	**1,637,000**	**1.64**
7	**Sewage pumping station**					
I	Wet well Cost	Sft	14,000	1,200	16,800,000	
Ii	Machinery Cost				264,000,000	
	Total Cost of Sewage Pumping Station				**280,800,000**	**280.80**
8	**Miscellaneous**	Lumpsum			**5,000,000**	**5.00**
Total Cost					**3,267,188,000**	**3,267.19**

12.4 WWTP MIR SHIKARA

Design discharge = 76,905 m^3/day (31 cusec)

Per capital sewage flow = 0.18 m^3/day

Population equivalent = 76,905/0.18=427,250 persons

Design calculations

Flow for one module = 12,000 m^3/day

Parameter		Unit	Quantity
Flow of one module	Q	m^3/day	12,000
Population served by one module	P	persons	66,667
Influent BOD	Li	mg/L	250
Design Temperature (mean temp of coldest month)	T	°C	16
ANAEROBIC POND			
Design Criteria			
The design of anaerobic ponds is based on the following parameters:			
Detention time		days	1 - 5
Solid accumulation rate (SAR)		m^3/person/yr	0.03 - 0.04
Depth		m	3 - 4.5
Allowable BOD loading λv=(20T-100) when T=10-20°C			
Allowable BOD loading λv=(10T+100) when T=20-25°C			
BOD removal in Anaerobic pond (2T+20)			
(**Reference:** Duncan Mara (1997), Design Manual For Waste Stabilization Ponds)			
Design Calculation			
Allowable BOD loading λv=(20T-100) when T=10-20°C		g/m^3.day	220
Volume of anaerob c pond: $\lambda_v = \frac{L_i Q}{V}$	V	m^3	13,636
Actual detention Time	Dt Va/Q	days	1.14
Adopted detention time for better efficiency	Dta	days	2

Parameter			Unit	Quantity
Adopted volume of anaerobic pond	Va		m^3	24,000
Sludge Accumulation rates (SAR)			m^3/person/yr	0.03
Volume of sludge accumulation per year	V1	P x SAR	m^3	2,000
Let Frequency for desludging @ 3 year	V3	V1 x 3	m^3	6,000
Total Volume (Vt)	Vt	V3+ Va	m^3	30,000
Let wastewater depth in Anaerobic Ponds		D	m	4.0
Free Board			m	1.0
Total Depth			m	5.0
Mid depth area of Anaerobic ponds	Am=	Vt/D	m^2	7,500
Take side slopes as 1 Vertical : 3 Horizontal				
Removal efficiency= 2T+20			%	52.00
Effluent BOD from anaerobic pond			mg/L	120.00
Mid Depth Area of Pond	A1	V/d1	m^2	7,500
Adopting Water Depth of pond	d1		m	4.00
Free Board	d1f		m	1.00
Total Depth			m	5.00
L:W Ratio			r	2.00
Mid Depth Length	Lm	$\sqrt{(Am/r)}$	m	61.24

Parameter			Unit	Quantity
Mid Depth Width	Wm	Lm x r	m	122.47
Top Length	Lt	Lm +18	m	79.24
Top Width	Wt	Wm +18	m	140.47
Bottom Length	Lb	Lm -12	m	49.24
Bottom Width	Wb	Wm - 12	m	110.47
Top Area of Anaerobic Pond	At	LtxWt	m^2	11,131
	At		Hectare	1.11
BOD removal in Anaerobic Pond (2T+20)		%	52	
BOD after treatment in Anaerobic pond			mg/L	120.0
FACULTATIVE POND				
Design Criteria				
The design of Facultative ponds is based on the following parameters:				
Detention time			days	5-12
Depth			m	2 - 2.5
Side slopes			Vert:Horiz	1:3
Allowable BOD loading λs=(20T-60) when T=10-20°C			Kg/hectare.day	
BOD removal in Facultative pond is given by following formula				
Effluent BOD		Le=	$\frac{L_i}{(1 + Kt)}$	

Parameter			Unit	Quantity
Where				
K = BOD removal rate constant			per day	0.2
t = Detention time			days	
Design Calculation				
Design wastewater flow	Q		m^3/day	12,000
Allowable Surface loading rate λs = (20T-60)	λs		kg/ha.day	260
Influent BOD for Facultative ponds (Li)	Li		mg/L	120.0
Mid depth area of Facultative pond Af = (10LiQ)/ λs	Af		m^2	55,385
Take wastewater depth of faculatative pond	Df		m	2.00
Free Board			m	1.00
Total Depth			m	3.00
Volume of facultative pond	Vf	AfxDf	m^3	110,769
Actual detention time in facultative pond	Dt	Vf/Q	days	9.23
Top Width	W2t	Wt	m	140
Mid Depth Width	W2m	W2t-12	m	128
Mid depth length	L2m	Af/W2m	m	431
Top length	L2t	L2m + 12	m	443
Bottom width	W2b	L2m - 6	m	122
Bottom length	L2b	L2m-6	m^2	425
Total area of Facultative pond	WtxLt		m^2	62,243
			Hectare	6.2

Parameter			Unit	Quantity
BOD removal in facultative ponds				
BOD of the finally treated wastewater	Le	=	$\frac{L_i}{(1+Kt)}$	
	Le	=	mg/L	42
Effluent BOD Standard in Pakistan		**=**	**mg/L**	**80**
Total area of Anaerobic and Facultative ponds of one module		=	Hectare	7.3
Add 50 % more for bunds, buffer zone and services		=	Hectare	3.67
Total area required for ONE MODULE of WSP system		=	Hectare	11.01
Design discharge (2038)		**Qd**	**m^3/day**	76,905
No of modules required for WWTP		Qd/Q		6.4
			say	7.0
DESIGN SUMMARY				
Effluent BOD		=	mg/L	42
Top Length of Anaerobic Pond		=	m	79.0
Top Width of Anaerobic Pond		=	m	140.0
Total Depth of Anaerobic Pond		=	m	5.00
Top Width of Facultative Pond		=	m	140.0
Top Length of Facultative Pond		=	m	443.0
Total Depth of Facultative Pond		=	m	3.00

12.4.1 Drawing for WWTP Mir Shikara

The drawing are attached at the end of this report.

12.4.2 Cost of WWTP Mir Shikara

The approximate cost is shown in Table 12.3

Table 12.3: Rough cost estimate for WWTP Mir Shikara

Sr. No.	Description	Unit	Qty.	Rate (Rs.)	Amount (Rs.)	Amount (in Million Rs.)
1	**Waste Stabilization Ponds**					
	Cost of 1 module					
I	Excavation	Cft	3,788,210	1.76	6,660,502	
Ii	Filling	Cft	756,551	1.37	1,034,586	
iii	Internal Lining	Sft	180,049	105.25	18,949,868	
iv	Outer Lining	Sft	71,638	105.25	7,539,856	
v	Bed Area	Sft	619,014	105.25	65,150,516	
vi	Brick lining at Top	Sft	96,977	105.25	5,405,124	
vii	Railing	Rft	4,351	55.74	1,296,828	
viii	Toe Length	Rft	4,641	298.08	2,121,828	
Ix	Drains	Rft	6562	1905	12,500,000	
					120,659,000	
	Total Cost of 7 Modules				**844,613,000**	**844.62**
2	**Cost of land**	Acre	282	700,000	**197,400,000**	**197.40**
3	**Site External Works**					
I	Approach Road	1000 Rft	2.30	915,000	2,101,000	
Ii	Track	1000 Rft	4.13	1,000,000	4,133,000	

Sr. No.	Description	Unit	Qty.	Rate (Rs.)	Amount (Rs.)	Amount (in Million Rs.)
iii	Fencing	Rft	14,140	153	2,163,500	
Iv	Parking Area	Sft	2,000	85	170,000	
V	Sewerage	Lumpsum			2,000,000	
vi	Electrical	Lumpsum			1,881,000	
vii	Piping Cost	Rft	2,000	800	1,600,000	
viii	Gate	No.	1	120,000	120,000	
	Total Cost of Site External Works				**14,168,500**	**14.17**
4	**Office Block**	Sft	2,336	1,580	**3,690,000**	**3.69**
5	**Staff Quarters**	Sft	700	1,395	**976,000**	**0.98**
6	**Store / Shed**	Sft	2,336	1,395	**1,637,000**	**1.64**
7	**Sewage pumping station**					
I	Wet well Cost	Sft	4,375	1,200	5,250,000	
Ii	Machinery Cost				84,000,000	
	Total Cost of Sewage Pumping Station				**89,250,000**	**89.25**
8	**Miscellaneous**	Lumpsum			**5,000,000**	**5.00**
Total Cost					**1,156,735,000**	**1,156.74**

12.5 WWTP ADU RAI DRAIN

Design discharge = 9670 m^3/day (4 cusec)

Per capital sewage flow = 0.18 m^3/day

Population equivalent = 9670/0.18=53,722 persons

Design Calculations

Flow for one module = 12,000 m^3/day

Parameter			Unit	Quantity
Flow of one module	Q		m^3/day	12,000
Population served by one module	P		persons	66,667
Influent BOD	Li		mg/L	250
Design Temperature (mean temp of coldest month)	T		°C	16
ANAEROBIC POND				
Design Criteria				
The design of anaerobic ponds is based on the following parameters:				
Detention time			days	1 - 5
Solid accumulation rate (SAR)			m^3/person/yr	0.03 - 0.04
Depth			m	3 - 4.5
Allowable BOD loading λv=(20T-100) when T=10-20°C				
Allowable BOD loading λv=(10T+100) when T=20-25°C				
BOD removal in Anaerobic pond (2T+20)				
(**Reference:** Duncan Mara (1997), Design Manual For Waste Stabilization Ponds)				
Design Calculation				
Allowable BOD loading λv=(20T-100) when T=10-20°C			g/m^3.day	220
Volume of anaerobic pond: $\lambda_v = \frac{L_i Q}{V}$	V		m^3	13,636
Actual detention Time	Dt	Va/Q	days	1.14
Adopted detention time for better efficiency	Dta		days	2

Parameter			Unit	Quantity
Adopted volume of anaerobic pond	Va		m^3	24,000
Sludge Accumulation rates (SAR)			m^3/person/yr	0.03
Volume of sludge accumulation per year	V1	P x SAR	m^3	2,000
Let Frequency for desludging @ 3 year	V3	V1 x 3	m^3	6,000
Total Volume (Vt)	Vt	V3+ Va	m^3	30,000
Let wastewater depth in Anaerobic Ponds		D	m	4.0
Free Board			m	1.0
Total Depth			m	5.0
Mid depth area of Anaerobic ponds	Am=	Vt/D	m^2	7,500
Take side slopes as 1 Vertical : 3 Horizontal				
Removal efficiency= 2T+20			%	52.00
Effluent BOD from anaerobic pond			mg/L	120.00
Mid Depth Area of Pond	A1	V/d1	m^2	7,500
Adopting Water Depth of pond	d1		m	4.00
Free Board	d1f		m	1.00
Total Depth			m	5.00
L:W Ratio			r	2.00
Mid Depth Length	Lm	√(Am/r)	m	61.24

Parameter			Unit	Quantity
Mid Depth Width	Wm	Lm x r	m	122.47
Top Length	Lt	Lm +18	m	79.24
Top Width	Wt	Wm +18	m	140.47
Bottom Length	Lb	Lm -12	m	49.24
Bottom Width	Wb	Wm - 12	m	110.47
Top Area of Anaerobic Pond	At	LtxWt	m^2	11,131
	At		Hectare	1.11
BOD removal in Anaerobic Pond (2T+20)		%	52	
BOD after treatment in Anaerobic pond			mg/L	120.0
FACULTATIVE POND				
Design Criteria				
The design of Facultative ponds is based on the following parameters:				
Detention time			days	5-12
Depth			m	2 - 2.5
Side slopes			Vert:Horiz	1:3
Allowable BOD loading λs=(20T-60) when T=10-20°C			Kg/hectare.day	
BOD removal in Facultative pond is given by following formula				
Effluent BOD		Le=	$\frac{L_i}{(1 + Kt)}$	

Parameter			Unit	Quantity
Where				
K = BOD removal rate constant			per day	0.2
t = Detention time			days	
Design Calculation				
Design wastewater flow	Q		m^3/day	12,000
Allowable Surface loading rate λs = (20T-60)	λs		kg/ha.day	260
Influent BOD for Facultative ponds (Li)	Li		mg/L	120.0
Mid depth area of Facultative pond Af = (10LiQ)/ λs	Af		m^2	55,385
Take wastewater depth of faculatative pond	Df		m	2.00
Free Board			m	1.00
Total Depth			m	3.00
Volume of facultative pond	Vf	AfxDf	m^3	110,769
Actual detention time in facultative pond	Dt	Vf/Q	days	9.23
Top Width	W2t	Wt	m	140
Mid Depth Width	W2m	W2t-12	m	128
Mid depth length	L2m	Af/W2m	m	431
Top length	L2t	L2m + 12	m	443
Bottom width	W2b	L2m - 6	m	122
Bottom length	L2b	L2m-6	m^2	425
Total area of Facultative pond	WtxLt		m^2	62,243
			Hectare	6.2

Parameter			Unit	Quantity
BOD removal in facultative ponds				
BOD of the finally treated wastewater	Le	=	$\frac{L_i}{(1+Kt)}$	
	Le	=	mg/L	42
Effluent BOD Standard in Pakistan		**=**	**mg/L**	**80**
Total area of Anaerobic and Facultative ponds of one module		=	Hectare	7.3
Add 50 % more for bunds, buffer zone and services		=	Hectare	3.67
Total area required for ONE MODULE of WSP system		=	Hectare	11.01
Design discharge (2038)		**Qd**	**m^3/day**	9670
No of modules required for WWTP		Qd/Q		0.8
			say	1.0
DESIGN SUMMARY				
Effluent BOD		=	mg/L	42
Top Length of Anaerobic Pond		=	m	79.0
Top Width of Anaerobic Pond		=	m	140.0
Total Depth of Anaerobic Pond		=	m	5.00
Top Width of Facultative Pond		=	m	140.0
Top Length of Facultative Pond		=	m	443.0
Total Depth of Facultative Pond		=	m	3.00

12.5.1 Drawing for Adu Rai

The drawing are attached at the end of this report.

12.5.2 Cost of WWTP AduRai

The approximate cost is shown in Table 12.4

Table 12.4: Rough cost estimate for WWTP AduRai

Sr. No.	Description	Unit	Qty.	Rate (Rs.)	Amount (Rs.)	Amount (in Million Rs.)
1	**Waste Stabilization Ponds**					
	Cost of 1 module					
I	Excavation	Cft	3,788,210	1.76	6,660,502	
Ii	Filling	Cft	756,551	1.37	1,034,586	
iii	Internal Lining	Sft	180,049	105.25	18,949,868	
iv	Outer Lining	Sft	71,638	105.25	7,539,856	
v	Bed Area	Sft	619,014	105.25	65,150,516	
vi	Brick lining at Top	Sft	96,977	105.25	5,405,124	
vii	Railing	Rft	4,351	55.74	1,296,828	
viii	Toe Length	Rft	4,641	298.08	2,121,828	
Ix	Drains	Rft	6562	1905	12,500,000	
					120,659,000	
	Total Cost of 1 Module				**120,659,000**	**120.66**
2	**Cost of land**	Acre	41	1,000,000	**41,000,000**	**41.00**
3	**Site External Works**					
I	Approach Road	1000 Rft	3.30	915,000	3,001,200	
Ii	Track	1000 Rft	1.70	1,000,000	1,700,000	
iii	Fencing	Rft	6,322	153	967,300	
Iv	Parking Area	Sft	1,000	85	85,000	
V	Sewerage	Lumpsum			1,000,000	
vi	Electrical	Lumpsum			1,881,000	
vii	Piping Cost	Rft	1,000	800	800,000	
viii	Gate	No.	1	120,000	120,000	
	Total Cost of Site External Works				**9,555,000**	**9.56**
4	**Office Block**	Sft	2,336	1,580	**3,690,000**	**3.69**
5	**Staff Quarters**	Sft	700	1,395	**976,000**	**0.98**
6	**Store / Shed**	Sft	2,336	1,395	**1,637,000**	**1.64**

Sr. No.	Description	Unit	Qty.	Rate (Rs.)	Amount (Rs.)	Amount (in Million Rs.)
7	**Sewage pumping station**					
I	Wet well Cost	Sft	700	1,200	840,000	
Ii	Machinery Cost				12,000,000	
	Total Cost of Sewage Pumping Station				**12,840,000**	**12.84**
8	**Miscellaneous**	Lumpsum			**5,000,000**	**5.00**
Total Cost					**195,356,000**	**195.36**

12.6 SUMMARY OF COST FOR MULTIPLE WWTP

Table 12.5: Summary of cost for multiple WWTPs

Sr. No.	Name of WWTP	Amount (Rs Million)
1	QilaMian Singh	2187
2	Main Drain and Jinnah Drain	3267
3	Mir Shikara	1157
4	AduRai	195
	Total	**6806**

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Drawings