DESIGN OF SEWERAGE SYSTEM AND SEWAGE TREATMENT PLANT FOR A MEDIUM SIZE CANTONMENT

A

THESIS REPORT

TOWARDS THE PARTIAL FULFILLMENT FOR
THE AWARD OF DEGREE OF

MASTER OF ENGINEERING

IN

ENVIRONMENTAL ENGINEERING

BY

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UNDER SUPERVISION OF
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CERTIFICATE

This is to certify that thesis entitled "Design of Sewerage System and Sewage Treatment Plant for a Medium Size Cantonment" submitted by **Ajay Kumar Sharma** in partial fulfillment of the requirements for the award of degree of **Master of Engineering in Environmental Engineering**, to Thapar Institute of Engineering and Technology (Deemed University), Patiala is a record of student's own work carried out by him under my supervision and guidance. The report has not been submitted for the award of any other degree or certificate in this or any other University or Institute.

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Last but not least the words are insufficient to express the sincere gratitude to my family without whose cooperation, I would not have seen this day. Above all, I thank the Almighty GOD whose spiritual strength made me work during this time.

Ajay Kumar Sharma

DECLARATION

I undersigned hereby declare that the thesis report entitled **"Design of Sewerage System and Sewage Treatment Plant for a Medium Size Cantonment"** written & submitted by me to Thapar Institute of Engineering and Technology (Deemed University), Patiala, in partial fulfillment of the requirements for the degree of **Master of Engineering in Environmental Engineering**. This is my original work & conclusions drawn therein are based on the material collected by me.

I further declare that this work has not been submitted to this or any other university for the award of any other degree, diploma or equivalent course.

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LIST OF ABBREVIATIONS

ASP:	Activated Sludge processes
BOD:	Biological Oxygen Demand
CWS:	Constructed Wetland Systems
DO:	Dissolved Oxygen
FWS:	Free Water Surface
HRT:	Hydraulic Retention Time
IL:	Invert level
L/E:	Lower end
MH:	Manhole
MLSS:	Mixed liquor suspended solids
N:	Nitrogen
RBC:	Rotating Biological Contactors
SRT:	Sludge retention time
STP:	Sewage treatment plant
SVI:	Sludge volume index
TKN:	Total Kjeldahl Nitrogen
TP:	Total Phosphorous
TSS:	Total Suspended Solids
U/E:	Upper end
VSB:	Vegetated Submerged Bed

CONTENTS

CHAPTER 1

INTRODUCTION

1.1 Background

Safe water and hygienic sanitation facilities are the two basic essential amenities the community needs for healthy living. Hence these two facilities are gaining importance in the country specially for medium size towns, satellite communities and rural human settlements. Though urban areas are being provided with sewerage systems and sewage treatment plant for safe disposal municipal sewage, rural areas and satellite communities still lack this basic need.

This thesis is an effort to design an appropriate sewerage scheme for a medium size cantonment, keeping in view the typical requirements and problems related with scattered areas, thin population density and lack of skilled manpower and resources. In this thesis an attempt also has been made to provide an alternative to conventional sewer and sewage treatment plant in the form of small bore system and constructed wetland system respectively. Since scarcity of water for human use becoming a global problem specially in developing countries like India, where resources are limited and being exploited inefficiently apart from spurge in population, reuse of treated wastewater becomes a preferred disposal method rather than discharging it to the inland water bodies. Keeping this in view this work takes into consideration reuse of treated effluent for irrigation of golf course and open green areas where water requirement is more and problem of distribution of treated wastewater is less (being concentrated areas).

Deeply involved in the problems of sewage collection, treatment and disposal of a cantonment area, where frequent clogging and unsatisfactory sewage treatment provides a big challenge to any engineer, this work of thesis has been selected as an effort to suggest a scheme and design which can overcome these problems and provide a reasonably better solution to these challenges.

1.2 Relevance and Importance of This Thesis Work

There are around two hundred medium size cantonments in the country, most of these are located in the suburban, rural and remote areas. Their sewage management systems are not very satisfactorily functioning. Most of the cantonments have no sewage treatment plants. In the cantonments where they are present, they are failures and performing quite unsatisfactorily. In many of the cantonments sewage system are represented by community septic tanks with clarified sewage being discharged in to the near by natural drains, or to storm water sewers, or being disposed of through unplanned irrigation or by flooding the surrounding open areas.

These days under the infrastructural development program of cantonments focus on sewerage system and sewage treatment plant has immensely increased. All the cantonments are in fact going for the design, construction and operation of satisfactorily functioning sewage management system. In many of the cases local enforcement authorities and pollution control agency have been responsible for this increased focus on sewage management system. However, the cantonment management in most of the cases has been found handicapped by the non availability of satisfactory schemes and treatment technologies for sewage management.

The student has been associated with the sewage management systems of cantonments for the last 14 years and he is sufficiently conversant with the problems faced and with the desirable aspects whose incorporation is believed to improve the overall performance of the sewage management system. In the present study the student has used his experiences and reviewed the technologies that can be used while deciding the scheme for sewage management.

On implementation of the scheme developed and the system designed in the present study can prove very useful specially as demonstration/experimentation systems. The experiences gained with this system for the cantonment in question can be used in other cantonments for better implementation of system proposed and designed.   .

1.3 Overview of Contents

Contents of this thesis have been organized into the following eight chapters:

1. **Introduction**: This chapter introduces the topic and objective of this thesis. Relevance of thesis topic has also been briefly mentioned in this chapter. It further includes a brief overview of the contents of this report.

2. **Literature review**: Review of literature related to the topic has been presented in this chapter. This review has been arranged firstly for sewerage system and sewage treatment plant in general and subsequently to each of their sub systems. Being a

design project established procedures and criteria given in the hand books, text books and manuals have been given preference over the research papers. However in case of CWS since there is still no consensus on any particular procedure or criteria, emphasis on publish research and experiences elsewhere are given importance.

3. **Approach Followed**: This chapter discusses the step wise approach followed to achieve the objective of this work. This also lists out the sources of back ground information and data.

4. **Site Description**: This chapter gives brief overview of the cantonment and further provides background information regarding location, land use pattern within the cantonment and in the surrounding areas, hydrological and climatological data, and data related to water supply, etc. It further provides information on substrata and lists general considerations and requirements to be considered in this project.

5. **Scheme for Collection, Treatment and Disposal of Sewage**: This chapter emphasizes on selection of appropriate sewerage scheme for the cantonment in question. A brief discussion on various options considered and selected, with reasons are given in this section. Based on this discussion an appropriate scheme for sewerage system and for sewage treatment plant has been finally conceptualized and presented in this chapter.

6. **Design of Sewerage System**: This chapter includes complete design details the sewerage system, which includes building sewers, intercepting tanks and small bore sewer system. Design considerations, criteria and equations and design calculations are presented here.

7. **Design of Sewage Treatment Plant (STP) and Disposal System**: This chapter of thesis gives design details of the STP and disposal system. It also includes design considerations, criteria and equations, and design calculations. As a closing remark this chapter includes a brief discussion about the inappropriateness of ASP for present case and design of an alternative to ASP i.e. design of CWS as a better option is also included in this chapter.

8. **Conclusion**: This chapter outlines the conclusion drawn from this design work. It infers that although the system has been designed for smooth functioning with least maintenance, may have problems may be much more serious if the intercepting tanks

are not properly managed. It further concludes that small bore sewerage system will require a different set of equipments to maintain this. This suggest that this scheme can be an experimental site and can be used for organized and structured data collection which can be used for designing, operating and controlling of constructed wetland system in the country.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

The sewerage and sewage treatment system consists of various sub systems and each sub system is required to be designed in detail keeping in view the objective, and data and back ground information available. This chapter is an effort to review the literature relevant to design of sewerage system and sewage treatment plant for the present case. Being a design project established procedures and criteria given in the hand books, text books and manuals have been given preference over the research papers. However in case of CWS since there is still no consensus on any particular procedure or criteria, emphasis on publish research and experiences elsewhere are given importance.

2.2 Sewerage System

(i) <u>General considerations in planning, design and operation of sewerage system</u>: Planning and design considerations play an important role in design of sewage collection and treatment system. CPHEEO (1993) discusses these aspects in detail and suggests the design period for sewerage system and sewage treatment plant as 30 years except for pumping system which is for 15 years. It further suggests the method of forecasting the population and sources of wastewater inflow and recommends that wastewater can be considered as 80–90% of water supplied. It also mentions the engineering, environmental, process and cost considerations and procedure for preparation of feasibility and project reports. This manual also discusses constructional and maintenance aspects.

(ii) <u>Sewers</u>: Sewer conveys the wastewater from house to the sewage treatment plant. Metcalf and Eddy, Inc. (1981) reviews the hydraulics applicable to the sewer designs and mentions the modified equations applicable to pressure as well as gravity flow sewerage system. It suggests use of Manning’s equation for open channel flow to design the gravity sewer running full and partially full. Use of nomograms based on this equation also has been illustrated. This further elaborates on values of friction coefficient to be considered in the design and suggests that a minimum value of 0.013 and 0.015 shall be considered for new sewers and analysis of old sewers respectively. CPHEEO (1993) manual classifies the sewers into sanitary sewer, storm sewer and combined sewer. It suggests further classification of sanitary sewers into conventional and small bore systems. This manual also recommends the minimum size and slopes of sewer. The maximum and minimum velocity in the sewer has been suggested as 0.6 m/s and 3 m/s for conventional and 0.3 m/s and 3 m/s for small bore system respectively (Metcalf and Eddy, Inc. (1981) and CPHEEO (1993). Peavy *et al*. (1986) lists out the basic design considerations for the sewer and recommends a minimum slope of 0.0008 as practical slope for construction.

(iii) Intercepting tank (Septic tank): IS 2470 (Part-1)-1985, lays down the design aspects to be considered for septic tank. It recommends the maximum flow to the tank based on number of fixtures discharging simultaneously. This IS further elaborates the construction details and suggests typical sketches of septic tanks. It also tabulates the typical recommended sizes of septic tank based on number of persons. IS: 2470 (Part-2)-1985, provides information on treatment and disposal of septic tank effluent, the recommended methods of disposal are soil absorption system, biological filters and up flow anaerobic filters. Salvato (1982) in his book 'Environmental engineering and sanitation' reports BOD and TSS removal of around 60% and 50-70% respectively in the septic tanks. It further elaborates design and constructional details of septic tanks.

2.3 Sewage Treatment System

(i) Sump and Pumps

Maximum retention time in the sump is recommended as 10 minutes to avoid odor and septicity (Metcalf and Eddy, Inc., 1981). Design and selection of pump, location and layout of pump house has also been discussed in detail in this book. Further it provides detailed discussion on internal layout of the pump house emphasizing the need for making suitable provision for future expansion. Retention time is suggested as 10-30 minutes in CPHEEO (1993). This manual further discusses the geometry and construction details of sump.

(ii) Selection of Treatment system: Peavy *et al*. (1986) and Metcalf and Eddy, Inc. (2003) enumerates the advantages and limitations of various options for secondary treatment of sewage considered in this design under their respective topics and recommends aerobic treatment over anaerobic system, where low strength effluent like sewage is required to be treated.

(iii) Sewage treatment system: The secondary treatment of clarified sewage is designed to be accomplished by activated sludge process and constructed wetland system in the present case.

(a) Activated sludge plant: Ramalho (1983) elaborates the design procedure for activated sludge plants and lays down design criteria. It also discusses various modifications to this system. The fundamentals of this treatment system including biokinetics involved and experimental determination of bio kinetic parameters has also been discussed in detail. The design criteria for Indian condition have been reported in the CPHEEO (1993) manual. It also states the equations applicable and illustrates the recommended design graphs. Design criteria for secondary clarifier and constructional details have also been discussed in this literature.Metcalf and Eddy, Inc. (2003) provides information in detail about constructional and design aspects, including suggested criteria. It further illustrates the activated sludge plant and secondary clarifier details.

(b) Vegetated submerged bed (VSB) constructed wetland system:

A. General Considerations

EPA (1999) sees a great potential in CWS for secondary and tertiary treatment of low strength sewage like municipal sewage. EPA (1995) discusses in length about general considerations in using CWS, it further lists out advantages and disadvantages of this system. Denny (1997) recommends that CWS should be considered as an entire package which apart from meeting its prime objectives of treatment should provide for sound foundation for nature conservation and biomass production.

B. Classification of CWS System:

EPA, 1999 based on the location of hydraulic grade line classifies CWS into two categories as free water surface (FWS) constructed wetland system and vegetated submerged bed constructed wetland system. Brix (1993) further classifies FWS based on types of macrophytes into free floating system, rooted emergent macrophytes system and submerged macrophyte system. Kadlec (1995) has classified constructed wetland systems into surface flow systems, vegetated subsurface flow systems, submerged aquatic bed systems and floating leaved aquatics systems. Wood (1995) finds that surface flow constructed wetland systems are popular in United States, particularly for large wastewater flows and for polishing and nutrient removal, whilst subsurface flow systems are widely accepted throughout Europe, Australia and South Africa

C. <u>Pollutant Removal Mechanism</u>

Wastewater treatment in CWS is accomplished through combination of biological, physical and chemical interactions between wastewater and plants, substrata and microbial community of the system (Wood, 1995). Kadlec and Reddy (2001) reports the temperature effect on performance of CWS and infers that temperature affects the performance of these systems being predominantly biological. Gearheart *et al*. (1989) confirms that pollutant removal is better in planted cells in comparison to unplanted cells. Perdomo *et al.* (1999) reports that emergent macrophytes results in better removal efficiencies than floating species. Although it is a general belief that plants may result in increase in hydraulic conductivity of bed media, but Brix (1997) based on data collected from Austria, Denmark and UK has reported contrary to this. He further reports data related to removal of nitrogen and phosphorous if biomass is harvested and on rate of release of oxygen from the roots. Brix (1994) tabulates the relative importance of the macrophytes in different designs of CWS. Reddy and D' Angelo (1997) discusses removal mechanisms in CWS at length for removal of various pollutants like BOD, TSS, TP, TN, etc. Although CWS has been found effective in pathogen removal (35-90%) as reported by Rivera *et al.,* (1995), but still it fails to meet stringent effluent standards for pathogens, disinfection may be required to prior to discharge of CWS treated effluents (Williams *et al.*, 1995).

D. <u>Design of CWS</u>

EPA, 1999 discusses in detail about performance of CWS with varying influent pollutant and presents the graphical compilation of data relating influent concentration vis-à-vis expected removal for all major pollutants Based on these studies it lays down the design criteria and procedures. It suggests a BOD and TSS areal loading rate of 60 Kg/ had and 200 Kg/ha.d respectively. These rates suggested by other researchers are 75 kg/ha.d and 200 kg/ha.d for BOD and TSS respectively (Wood, 1995 and Crites, 1994). Since these criteria are based on research for temperate climate higher of these two may prove more appropriate for tropical climate. EPA (1999) further discusses design considerations and presents empirical method for design of this system. Cooper and Green (1995) suggest an equation to calculate the surface area. Crites (1994) and EPA (1999) recommend that removal of BOD can be assumed to follow first order kinetics and suggested equations for this.

E. Performance evaluation

From the review of literature it is seen that very few constructed wetland system are in operation in India. Based on experiences with CWS at Bhubaneshwar, Juwarkar *et al.* (1995) reports removal efficiencies of 78–91% for BOD and finds it a cost effective alternative to conventional treatment system and recommends it for small towns and villages. Billore *et al.* (1999) based on a field scale study on VSB system report around 70% removal of BOD. Yang *et al.* (1995) based on studies on VSB constructed wetland system at Bainking village in China, treating 3100 m^3/d reports removal efficiency for BOD and TSS as 90.5% and 92.6% respectively. Experience over 10 years in England has indicated general acceptability of constructed wetland system as an appropriate treatment for small communities (Cooper and Green, 1995). Xianfa and Chuncai (1995) reports that use of CWS is increasing in China in recent years and researchers in China are greatly interested in integrated system. EPA (1993) presents a technology assement on these systems and find it as an appropriate treatment system.

2.4 Disposal System

Metcalf and Eddy, Inc. (2003) discuss the feasibility, criteria, probable reuse of treated wastewater keeping public health and environmental issues in view. It favors the reuse of water for golf courses, green areas around buildings, parks etc. where irrigational water requirement is substantial. It also talks about the technologies for water reclamation and storage of reclaimed water. Environmental (protection) Rules, 1986 lays down the standards for wastewater for irrigation as well as discharging this into inland surface water bodies.

CHAPTER 3

APPROACH FOLLOWED

3.1 Introduction

Objective of the present study is to design a sewerage system and a sewage treatment plant for a medium size cantonment. Approach followed for the design of the above systems is briefly described in this chapter. The work was divided into the following phases:

- Collection of background information
- Analysis of the collected information for getting the basic information required in the design and preparation of site map with necessary details
- Finalizing the schemes for the sewerage system and sewage treatment
- Design of sewerage system
- Design of sewage treatment plant

In the following paragraphs the approach followed for completing each of the phases of work is briefly described.

3.2 Collection of background information

The following data was collected in this phase of study:

- Water supply details (quantity, patterns and schedules of supply)
- Typical characteristics of the sewage to be collected, treated and disposed
- Zoning details of the cantonment in question, current population distribution patterns and future plans of growth
- Location details, land use patterns and drainage patterns of the area in which the cantonment is located
- Land survey details of the cantonment
- Requirements of the environmental law to be complied with
- Availability of power (and its dependability)
- Requirements of the cantonment management to be satisfied
- Weather conditions of the area
- Sub soil conditions, and ground water and hydrology details

The required information was collected from the maps and data available with local military authorities; topo-sheet of the cantonment prepared by the Survey of India; weather details from the district administrative authorities and from the web site of the Indian Meteorological Department. To supplement the data obtained the site was physically surveyed and relevant officials were discussed with by the student.

For typical characteristics of sewage, sewage characteristics from four different cantonments were collected. Depending on the conditions unique to the cantonment in question necessary adjustments were made.

3.3 Data Analysis and Obtaining the Basic Information Required

On the basis of the analysis of the data collected the following have been done:

(i) Population to be served and its distribution pattern had been marked on the map(both present and design period scenarios)

(ii) Potential sites for STP had been identified

(iii)Potential for recycling and reuse of treated effluent and areas where treated effluent can be reused had been assessed

(iv)Site map showing the details such as, road network, layout of buildings, design population distribution, natural drains, contour details, boundaries, and land use patterns of adjoining areas had been marked on the site map

3.4 Finalizing the Schemes for the Sewerage System and Sewage Treatment

Based on the background information collected, while giving due importance to the requirements of the management and environmental law to be satisfied, in the light of literature review, schemes for sewerage system and for the sewage treatment plant were finalized.

Some of the aspects given due importance in deciding the scheme for sewerage system are:

(i) Preventing clogging of sewers and minimizing need for frequent cleaning

(ii) Minimizing the excavation costs and eliminating the need for intermediate pumping of sewage

Similarly the aspects considered in deciding the scheme for sewage treatment and disposal are

(i) Ensuring operational ease and elimination of need for trained manpower

(ii) Minimizing the need for conventional energy resources like electricity

(iii) Minimizing or eliminating generation of STP sludges

(iv) Ensuring social acceptability and overall aesthetics

(v) Production of byproducts and resources from sewage treatment

The finalized scheme was marked on the site map mentioned earlier.

3.5 Design of Sewerage System

Layout of sewers, location of intercepting tanks, population served by each of the intercepting tanks are indicated in the layout map in different colors. Further, all stretches of sewers and intercepting tanks were identified by unique identification codes. This was felt needed for avoiding confusion and complexities.

Sewerage system was considered to include the following three components:

(i) Design of building sewers

(ii) Design of intercepting tanks

(iii) Design of small bore sewerage system

The building sewers are supposed to carry the sewage to the local intercepting tank located with 80 meters distance from the house connection. Design of building sewers was intentionally not included in the present study. However, for getting the invert levels for the design of intercepting tanks and small bore sewerage system, building sewer related to the critical intercepting tank was designed.

Intercepting tanks were supposed to be similar to the conventional septic tanks. Population to be served by each of the intercepting tanks was calculated and indicated in the study. Using this information, typical designs and sketches for the intercepting tanks can be obtained from the relevant IS code.

Small bore sewerage system was designed while using the technique described in "Waste water engineering: Collection and pumping of wastewater" by Metcalf and Eddy, Inc., (1981). Manning's equation and hydraulic relationships for circular pipes were used in the design of sewers. Since flows in the sewers were less, the nomograms based on Manning's equation available for simplifying the design could not be used in the present design. All design calculations were made manually with calculator.

3.6 Design of Sewage Treatment Plant (STP)

Sewerage system according to the scheme finalized is supposed to have the following:

(i) Clarified sewage sump and pumps

(ii) Secondary treatment system (either activated sludge treatment plant or vegetated submerged bed constructed wetland system)

(iii) Fish pond and treated effluent disposal system

3.6.1 Sump and pump: Capacities of sump and pump were worked out on the basis of the assumptions, such as the following:

(i) The sewage is to be pumped through the STP three times a day coinciding with water supply schedule

(ii) 60% of the total discharge was assumed to be received in the morning and required to be pumped through the STP in 3 hours time

(iii) Reliability of pumping while keeping in mind the pump failure and power failure

3.6.2 Secondary treatment unit: Characteristics of the sewage to be treated in the secondary treatment system was worked out through assuming typical removal rates in the intercepting tanks (indicated in the literature).

(i) **ASP:** Activated sludge treatment plant was designed while following the method given in the "CPHEEO Manual on sewerage and sewage treatment plant (1993)" and "Introduction to wastewater treatment processes" by Ramalho (1983).

(ii) **VSB:** Vegetated submerged bed constructed wetland system was designed on the basis of the design criteria and equations given in EPA manual. The design was then cross checked with criteria given in a few other published works.

(iii) **Fish pond and treated effluent disposal system:** Treated effluent was required to be hold in a fishpond for 10 days. Irrigation requirements of water are supposed to be met through pumping the effluent from the fishpond as and when required. Excess treated effluents were supposed to be over flown into a nearby natural surface drain.

CHAPTER 4

SITE DESCRIPTION

4.1 Background Information

A medium size cantonment can be taken as the habitat having population around 4000 to 6000. This is an area having definite boundaries and comprises of broadly following types of buildings:

(i) Residential Quarters for families.

(ii) Bachelors accommodation with attached toilets

(iii) Barracks (hostel type accommodation) with community kitchen and toilets

(iv) Office accommodation

(v) Hospital ($\leq$ 50 bedded)

(vi) Garages and work shops

(vii) Small shopping centre

(viii) Small school

The cantonment for which sewerage system is designed has a design population of 4100. Since the future growth plan for the cantonment has already been laid down, standard methods of computing future population are not needed. Planned population size of the cantonment is 5800.

4.2 Location

The cantonment is surrounded by town on two sides and other two sides are having a highway and agricultural fields respectively. The town is having its own sewerage system and would nothing to do with that of the cantonment. hence this sewerage system is required to be planned independently and in totality. Detailed plan showing buildings, pocket wise population, storm water drains and roads lay out of cantonment drawn to a scale of 1:2000 (presented in Appx 'A') has been used for this design. The general topography of the area is ascertained from the Survey of India contour map (specially prepared for cantonment). It is seen from this that the area is generally flat with three localized depressions which can be utilized to create a pond. The soil strata is mainly comprising of clayey/silty soil and hence having very low percolation rate.

4.3 Hydrological and Climatological Information

The annual rainfall data are presented in the Table 2.1. From the table it can be seen that maximum rainfall is during the month of July-August and maximum daily rainfall is 70.2 mm. The total average rainfall is around 400 mm (based on information available with army authorities.

Since the daily record of minimum and maximum temperature is not being maintained by the district/army authority the temperature data of near by city (Table 4.2) has been considered in the present study. As per this record minimum and maximum monthly average temperature are 6.1^0C and 38.6^0C respectively.

Ground water table in this area is around 6.5 m. The cantonment is having an independent storm water collection and disposal system of open drains. The collected water is discharged into a local natural drain.

4.4 Water and Electric Supply

The cantonment is having its own water and electric supply distribution system. It is designed to cater 200 l/capita.day (lpcd) of water. The water is supplied three times a day during the following timings:

- Morning Supply 0500 h – 0830 h
- Afternoon supply 1230 h – 1330 h
- Evening supply 1700 h – 2030 h

However continuous water supply is ensured to individual buildings by providing service tanks. Keeping in view the water supply hours and life styles sewage flow can be assumed to peak during 0600 h to 0900 h and 1730 – 2030 h. The electric supply is erratic with on an average break down/shut down of 3-4 hours.

4.5 Typical Characteristics of Sewage

Typical characteristics of sewage have been decided based on the analysis report of sewage from four different cantonments. The degree of treatment required has been decided based on the discharge standards as laid down in the Environment (Protection) Rule (1986).

4.6 General Considerations

The sewage collection, treatment and disposal system as far as possible should be able to meet the following requirements:

(i) The system should have minimum line clogging problem to avoid frequent cleaning and maintenance.

(ii) Sewers should be laid along the roads for easy accessibility.

(iii) The system should have least dependence on electricity and mechanical devices like pumps etc.

(iv) It should have least requirement for skilled manpower and make use of locally available resources.

(v) System should be designed in such a way that it should perform satisfactorily at design load as well as present load.

(vi) It should be economical to construct, operate and maintain.

(vii) Treatment plant should be able to meet stringent effluent standards (present as well as expected in future) and should cater for temporary alternate system (during maintenance period).

(viii) Possibility to reuse the treated water.

Table 2.1: Annual rainfall data (based on the records held with district authorities)

Year	Month	Date	Rainfall (in mm)
2003	Jan	-	-
	Feb	19	Meager could not be recorded
		22	- do -
		28	- do -
	Mar	1	- do -
		2	- do -
	Apr	19	- do -
	May	23	Very low
	Jun	23	- do -
		30	- do -
	July	5	Meager could not be recorded
		11	4.8
		18	61.6
		19	70.2
		21	68.2
		22	3.7
		28	5.5
	Aug	2	20
		4	5
		5	11.4
		9	2.4
		11	4
		15	4.8
		22	42.4
		28	12.6
	Sep	-	-
	Oct	-	-
	Nov	17	Meager could not be recorded

	Dec	-	-
2004	Jan	21	9.4
		22	2.2
		30	3.6
	Feb	-	-
	Mar	-	-
	Apr	23	3.6
		30	12
	May	1	27.6

(Source: Data from District Authorities)

Table 2.2: Monthly minimum and maximum temperature of a near by city

Month	Mean Temperature ^{o}C Daily Minimum	Daily Maximum	Mean Total Rainfall (mm)	Mean Number of Rainy Days
Jan	6.1	20.4	33.1	2.6
Feb	8.3	23.1	38.9	2.8
Mar	13.4	28.4	30.4	2.6
Apr	18.9	34.5	8.5	1.1
May	23.1	38.3	28.4	2.1
Jun	25.4	38.6	145.2	6.3
Jul	23.9	34.0	280.4	12.3
Aug	23.3	32.7	307.5	11.4
Sep	21.8	33.1	133.0	5.0
Oct	17.0	31.8	21.9	1.4
Nov	10.5	27.3	9.4	0.8
Dec	6.7	22.1	21.9	1.4

(Source: Indian Meteriological Department Web Site)

CHAPTER 5

SCHEME FOR COLLECTION, TREATMENT AND DISPOSAL OF SEWAGE

5.1 Introduction

Selection of suitable scheme is the first step in designing of sewerage system and sewage treatment system. Finalization of the scheme requires careful review of back ground information and field data apart from the overall objective. In this chapter various options for sewerage systems and its sub systems have been evaluated keeping in view the objective, the field data and background information of the site. Based on this a scheme appropriate for this cantonment has been finalized.

5.2 General Considerations in Deciding Appropriate Scheme for Cantonment

General considerations in finalizing the scheme are:

(i) Topography of the area to decide the sewerage system layout and location of sewage treatment plant (STP)

(ii) Influent characteristics and the level of treatment required to be met to decide on treatment facilities.

(iii) Availability of land and its use pattern, and population density

(iv) Exploitation of the potential for reuse/recycling of treated wastewater

(v) Social and cultural acceptability of the scheme chosen

(vi) Ease of operation and maintenance

(vii) Cost of construction, operation and maintenance

The sewerage system broadly consists of the following:

(i) Collection system including sewer, intercepting tanks, and sewer appurtenances

(ii) Sewage treatment system

(iii) Disposal system

5.3 Selection of Collection System

5.3.1 Sewer

Selection of appropriate sewers is very important step in the design of a sewerage scheme. The options considered for this are:

(i) Sanitary sewers (conveys sewage only), and

(ii) Combined sewer (conveys both sewage and storm water)

Since the cantonment under consideration is already having storm water collection and disposal system, sanitary sewer system in which no storm water is allowed have been considered. Another reason for having separate system for sewage and storm water is to avoid unnecessary overloading of the sewage treatment plant (STP) and the complications that arise from dramatic variations in wastewater flows.

The sanitary sewer can either convey raw sewage (conveying both solid and liquid) or clarified sewage (after primary treatment). Since the cantonment area will not be thickly populated and since for most of sewers during most part of the year self cleansing velocities can not be ensured, instead of going for conventional sewerage system (which conveys both liquid and solids that increases the risk of sewage clogging) small bore system (wherein clarified sewage is conveyed in the sewers) has been preferred. Small bore system has been following additional advantages over conventional system (CPHEEO, 1993):

- (i) Relatively cheaper because of flatter slopes required for sewers (low excavation costs and reduced need for intermediate pumping of sewage)
- (ii) Because of no/less suspended solids the STP will be simpler and tackling mainly the dissolved organic matter
- (iii) Generation and handling of primary treatment sludges is altogether eliminated (in smaller treatment plants handling and management of STP sludges poses serious problems)

5.3.2 Intercepting Tank

In small bore sewer system the sewage is required to be clarified before allowing it to enter the sewer. Various options considered for primary treatment are:

(i) Primary clarifier

(ii) Intercepting tank for individual house/block, and

(iii) Intercepting tank for cluster of houses (community tank)

Since primary clarifier defeats the very purpose of providing small bore system unless provided for small cluster of houses (considered not feasible) there by keeping the short length of building sewer. Further it involves day to day handling and disposal of primary sludges. Hence primary clarifier system has not been considered. Intercepting tanks for individual houses apart from being costly are difficult to maintain. Hence these have also been rejected in this scheme. Community intercepting tank in the form of septic tank has been considered appropriate for cantonments.

5.4 Sewage Treatment System

Clarified sewage can not be disposed of unless it is treated to meet the effluent standards, laid down by the regulating authorities. Secondary treatment systems considered for this scheme are

(i) Anaerobic treatment processes

(ii) Lagoons/ponds

(iii) Trickling filter

(iv) Rotating biological contactors

(v) Activate sludge plants

(vi) Constructed wetland system

5.4.1 Anaerobic Treatment

Anaerobic treatment option include up flow and down flow anaerobic filters, fluidized bed systems, up flow anaerobic sludge blanket reactors (UASB), anaerobic lagoons, etc.

Though sludge handling in these systems reduces to a great extent these systems are not considered appropriate for municipal sewage (low strength). These systems are difficult to maintain and operate and if upset restarting requires large start–up time. Further clarification of effluents from an anaerobic treatment unit in secondary clarifier is more difficult. Anaerobic treatment is a potential source of odors and corrosive gases.

According to Metcalf and Eddy, Inc., (2003) both aerobic and anaerobic processes would require almost the same amount of energy input if biodegradable COD concentration of wastewater is 1270 mg/l. At lower COD concentrations aerobic processes requires less energy while anaerobic treatment system requires more energy inputs. Wastewater strength and temperature greatly affects the economy and feasibility of anaerobic treatment. 35^0C Temperature is considered optimum for efficient and more stable anaerobic treatment. COD concentration greater than 1500 – 2000 mg/l is usually needed for providing sufficient quantity of methane for heating the wastewater. IS: 2470 (part-2)-1985 suggests upflow anaerobic filter or biological filter (trickling filter) for secondary treatment of septic tank effluent but since it is not an appropriate solution to the needs of the present case, anaerobic treatment process has not been chosen.

5.4.2 Ponds and Lagoons

A wastewater pond, alternatively known as stabilization pond/oxidation pond, consists of a large, shallow earthen basin in which wastewater is retained for long enough to provide time for natural purification processes to provide necessary degree of treatment. In ponds there is no control over the process variables. Ponds invariably have weed infestation which is very difficult to control. Lagoons are distinguished from ponds by having provision for artificial oxygen supply. Further these systems are highly climate dependent, and hence erratic effluent quality can not be ruled out. These systems suffer from problems of bad odors, and flies and mosquitoes. These systems despite being simple in construction and operation are not considered appropriate for present case. Aesthetics and biodiversity reasons also support the decision against ponds and lagoons.

5.4.3 Trickling Filters

This is an attached growth biological treatment system in which randomly packed solid media provides surface for biofilm growth. The wastewater is uniformly applied using a rotating arm and this water trickles down to the bottom duly treated where it is collected and discharged. Though this system requires less (but consistent) energy and is relatively simple in operation has not been considered suitable for this scheme due to frequent clogging problems and odor and fly nuisance in case of improper operation and maintenance. Further, requirement of continuous skilled supervision and frequent maintenance of moving parts discourages use of trickling filters.

5.4.4 Rotating Biological Contactors (RBC)

RBC includes closely spaced plastic or metal discs mounted on a common horizontal shaft rotating at 1 to 4 rpm. These discs are maintained partially submerged in clarified sewage. Although suitable for municipal sewage treatment, it is still not being widely used specially in areas where continuous electric supply can not be ensured as in the present case. Further, frequent breakdown due to excessive biofilm growth and poor maintenance (weekly lubrication is required to be applied) are common in this system. Apart from this continuous energy and skilled manpower requirement takes it out of consideration in present case.

5.4.5 Activated Sludge Plant

This technology is most widely used for secondary treatment of municipal sewage as it is a matured technology with sufficient long term experiences. Due to this, design criteria and equations, and other related information are sufficiently available. This system requires relatively less land.

Although widely used for the secondary treatment of municipal sewage this technology suffers from the following limitations/drawbacks:

(i) Causes frequent maintenance and operational problems

(ii) With low strength wastewater, like municipal sewage, it is very difficult to maintain desired MLSS concentration in the aeration tank. For low strength clarified sewage (say < 100 mg/l) wastages of biological solids in the secondary clarified effluents (40 to 70 mg/l) will be significant and maintaining high MLSS in the aeration tank is very difficult.

(iii) Operation and maintenance of ASP requires skilled manpower and the system depend heavily on conventional energy resources. Further, sludge produced by an ASP has poor settling and dewatering properties. Hence the secondary sludge handling and management poses problems.

Despite the problems ASP has been considered as one of the options for secondary treatment of clarified municipal sewage. ASP has actually been considered for making comparisons and stating that how the other alternative is superior to it.

5.4.6 Constructed Wetland Systems (CWS)

CWS are modified and/or simplified ecological systems engineered and articulated by man from non-wetland sites and managed for the purpose of wastewater treatment. Constructed wetlands are cost-effective and technically feasible approach for treating wastewater. These systems are less expensive to build and operate than other treatment options if land is not a limiting factor. Further to this it:

(i) Can tolerate fluctuations in flow and facilitate water recycling and reuse

(ii) Can be built to fit harmoniously into the landscape

(iii) Can provide numerous other benefits, such as wildlife habitat and aesthetic enhancement of open spaces

Since land availability is not a limiting factor and the water is proposed to be reused this system provides the most appropriate solution for this medium size cantonment. Further to this it provides additional benefits in the form of biomass, fish and aesthetics.

Limiting factors of this treatment system are that there is yet no consensus on the optimal design of wetland systems and very little information is available on their long-term performance. Still this system is believed to hold lot of potential and promise specially in tropical countries like India that to for satellite community, like cantonments.

Constructed wetland systems are of two types:

(i) Free water surface constructed wetland systems (FWS)

(i) Vegetated submerged bed (VSB) constructed wetland systems (water surface remains below the bed media)

Of these, when viewed from the angle of technological and economic feasibility, ease of harvesting and utilization of the harvested biomass, flies and mosquitoes breeding problem, vegetated submerged bed constructed wetland systems (VSB), planted with emergent macrophytes, provides a better option than the FWS.

5.5 Disposal System

Treated effluent though satiesfy the quality requirement for disposal into any water body or on land, the present scheme proposes to reuse this treated effluent for pisciculture, irrigation etc. In this scheme the treated effluent will be retained in a fish pond of sufficient storage capacity. From this pond water will be pumped and used for the irrigation of golf course and plantation. Surplus water when irrigation requirement are low will be allowed to overflow into road side drains for disposal. The fish pond is not proposed to be lined and considerable quantities of treated effluent will be lost through percolation and evaporation.

Sludge generation is a problem only if ASP is used. Sludge from the ASP is proposed to be disposed of using the sludge drying beds, and the sludge cake thus formed can be used as manure. Constructed wetland system will not be generating any sludge on regular basis. The only time when sludge can be expected is during occasional maintenance (may be once in 10 years).

The scheme chosen for sewage collection, treatment and disposal is shown in Figure 5.1.

5.6 Conceptualized Scheme of Sewerage System:

(i) Building sewers are used to convey waste water from building to the community septic tank.

(ii) The clarified effluent is conveyed to the collection sump using a network of branch sewers, main sewer and trunk sewers (small bore system).

(iii) Collection sump and pumps are provided at treatment plant site, before sewage treatment plant to pump the wastewater through the treatment plant.

(iv) It is proposed to use ASP for secondary treatment alternatively since it is felt that constructed wet land system (non conventional technology) provides a better option it has also been proposed as preferred option.

(v) Treated effluent is proposed to use for pisciculture and then subsequently for irrigation (after disinfection if required). The surplus water during lean period is proposed to be wasted through road side open drains to the inland water bodies

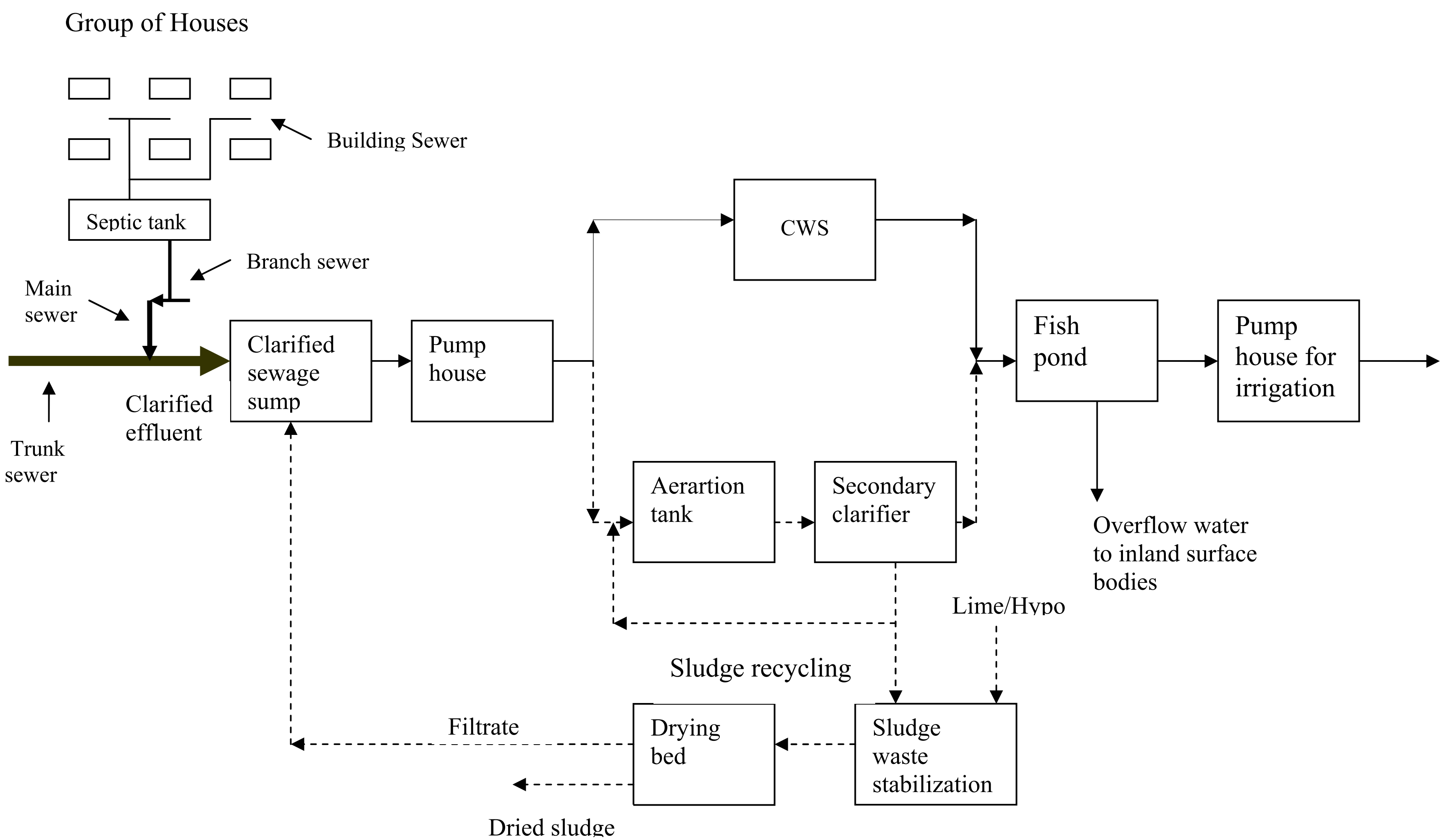

Figure 5.1: Proposed scheme of sewerage system and sewage treatment

CHAPTER 6

DESIGN OF SEWERAGE SYSTEM

6.1 Introduction

Satisfactory functioning of sewerage system requires careful consideration of all expected flows, variations in flow, operational and maintenance requirements, economics, structural soundness, etc. The sewer appurtenances shall be given due importance while designing the system. Small bore sewer system (which involves conveyance of clarified sewage) has been opted for this system. Design details of this system are presented in this chapter. Sewerage system in present case includes:

(i) Building and small bore sewer system

(ii) Intercepting tank

First step in the hydraulic design of a sewerage system is to prepare a map showing location of all sewer contributory area and population and proposed layout of sewers. A map showing the above details to a scale of 1 in 2000 has been used for this design and has been enclosed as Appendix ‘A’.

6.2 Design of Sewer

(a) Design Considerations

(i) The cantonment under consideration is already having a storm water collection and disposal system. Hence sanitary sewerage system in which no storm water is allowed has been considered. Another reason for having separate system for sewage and storm water is to avoid unnecessary overloading of the sewage treatment plant (STP) and the complications that arise from dramatic variations in wastewater flows to the STP.

(ii) Since the cantonment area will not be thickly populated and since for most of sewers during part of the year self cleansing velocities can not be ensured, instead of going for conventional sewerage system (which conveys both liquid and solids that increases the risk of sewage clogging), small bore system has been considered. In small bore system sewage from household is collected and clarified in intercepting tanks (septic tanks serving a group of closely located houses) and then allowed to enter the sewer. This system is supposed to have the following other advantages over the conventional system:

a) Relatively cheaper because of flatter slopes required for sewers. Further need for intermediate pumping of sewage is altogether minimized or avoided.
b) Because of no suspended solids the STP will be simpler and tackling mainly the dissolved organic matter.
c) Generation and handling of primary treatment sludges is altogether eliminated (in smaller treatment plants handling and management of STP sludges poses serious problems).

(iii) The quantity of sewage in the system is a function of use pattern and quantum of water supplied. All the water supplied is not collected as sewage from the household. Since some water is lost in evaporation, seepage into ground, leakages etc. only 80-90% of water supply may be expected to reach the sewer, further a sewer should be designed for a minimum flow of 100 liter per capita per day (CPHEEO, 1993). A significant quantity of water is lost in kitchen gardening, lawn irrigation, water sprinkling in open areas and other household uses where water is lost in evaporation. Hence only 80% of the supplied water has been considered as sewage generated by the households. Water supply in the cantonment is 200 lpcd (liter per capita per day). Hence, design sewage generation rate considered in the present study has been 160 lpcd. In the absence of exact data related to generation of wastewater from the community kitchens, dinning halls and toilets the discharge has been assumed as 100 lpcd.

(iv) Since population of the cantonment is 5800 for sewer design a peaking factor of 3 and self cleansing velocity of 0.6 m/s has been considered (CPHEEO, 1993). This peaking factor has been considered for the design of building sewers up to the intercepting tank. Beyond intercepting tank (for small bore system) a peaking factor of 2 has been considered. In small bore system, since clarified sewage is flowing, strict sewer gradients to ensure minimum self cleansing velocity are not necessary and hold no relevance. Still 0.3 m/s velocity at peak flow is recommended (CPHEEO, 1993) in sewers of even in small bore system.

(v) Minimum diameter of building has been considered as 150 mm and slope as 1 in 60, and for other sewer the minimum diameter and slope has been taken as 100 mm (CPHEEO, 1993) and 1 in 1250 (Peavy *et al.*, 1986).

(vi) No infiltration has been considered in this design since the complete collection system is above the ground water table all the time (maximum depth of sewer is 3.42 m, where as the ground water table is at 6.0 m. further with improved standard of workmanship and quality, and availability of improved construction aids infiltration if at all is likely to be eliminated.

(vii) For small bore system, cleanout have been considered instead of manholes except at major junctions. Cleanouts are required to be provided at all upstream ends, intersection of sewer lines, major changes in direction at high points and at an interval of 60 – 100 m in long flat sections.

(viii) Since the system is laid on a falling gradients and no humps are there vent cleanout to release air has not been considered.

(ix) While deciding about intercepting tanks the following aspects have been taken into account:

a) The plinth heights of the building are 450 mm from ground level. Assuming that water closet outlet is 200 mm below the plinth level and assuming invert level (IL) of first manhole (MH) from building as 250 mm below GL the hydraulic head of 0.5 m has been achieved for the building sewers. Maximum length of building sewer (from household intercepting tank) has been taken as 80 m.

b) For the first 20 m building sewer has been assumed to carry only 25 % of the estimated flow, for next 20 m the flow has been taken as 50 % and for the rest of the length 100% flow has been considered.

c) The invert level of (IL) outlet has been considered to be 0.05 m below the IL of inlet.

(b) Design Criteria and Equations

The hydraulic analysis of sewer is simplified by assuming steady flow conditions. The available head in wastewater line is utilized in overcoming surface resistance and in attaining kinetic energy for flow. The design practice is to use Manning's equation for open channel flow and Hazen-Williams and Darcy's-Weisback formulae for closed conduits or pressure flow(CPHEEO, 1993). The Manning's equation has been used for design in this document (being gravity flow) and can be expressed as:

$$V = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$$

Where V is velocity in m/s, n is coefficient of roughness, R is hydraulic radius in m, and S is slope of energy grade line (for uniform flow slope of channel bottom can be taken as slope). Hydraulic radius is defined as ratio of cross-sectional area of flow to the wetted perimeter. For a pipe flowing full the hydraulic radius is

$$R = \frac{\frac{\pi D^2}{4}}{\pi D} = \frac{D}{4}$$

Where D is diameter.

The typical n values for different pipe material is given in the literature and it varies from 0.011 to 0.015 for the pipe materials normally used i.e. salt glazed pipes, cement concrete pipes, plastic pipes etc. In this design the building sewer salt glazed pipes and in small bore sewer plastic pipes (high density poly ethylene pipes) are considered. For these materials n value is 0.013 and 0.011 respectively. It is recommended to use minimum n value of 0.013 for new sewer and 0.015 for analyzing the old sewer lines (Metcalf and Eddy, Inc., 1981). This value has been recommended considering the uncertainties inherent with sewer design and construction and the various joints in the line.

Since large number of calculations are involved in sewer design the nomograms for Manning's equation are generally used. However in this design a calculation approach has been followed since small discharges have not allowed use of nomograms. The shape of the pipes in this design are taken as circular and following formula has been used for determining the flow in partial as well as full running sewers (Metcalf and Eddy, Inc., 1981):

$$Q = \frac{k'}{n}(D^{8/3})(S^{1/2})$$

Where Q is flow rate in m^3/s, n is Manning's coefficient of friction, D is diameter in m, S is slope of energy grade line and k' is a fractional number. Hence for assumed diameter and slope, and known value of Q and n the value of k' can be determined, from this value d/D (ratio of depth of flow to diameter of pipe) can be determined using the Table 6.1. Value of d/D in turn can be used to find out the fractional area of flow using hydraulic elements graph for circular sewer (Figure 6.1). Using this area and discharge, velocity of flow in the pipe can be determined. Small bore system can be designed to flow full unlike the normal sewers. Since the complete pipe is of same material and not a composite pipe it has been assumed that n remains constant with depth.

Table 6.1: Values of k’ for circular channels in terms of diameter in the equation

$Q = (K'/n)D^{8/3}S^{1/2}$

$\frac{d^b}{D}$	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	....	0.000047	0.00021	0.00050	0.00093	0.00150	0.00221	0.00306	0.00407	0.00521
0.1	0.00651	0.00795	0.00953	0.0113	0.0131	0.0152	0.0173	0.0196	0.0220	0.0246
0.2	0.0273	0.0301	0.0331	0.0362	0.0394	0.0427	0.0461	0.0497	0.0534	0.0572
0.3	0.0610	0.0650	0.0691	0.0733	0.0776	0.0820	0.0864	0.0910	0.0956	0.1003
0.4	0.1050	0.1099	0.1148	0.1197	0.1248	0.1298	0.1349	0.1401	0.1453	0.1506
0.5	0.156	0.161	0.166	0.172	0.177	0.183	0.188	0.193	0.199	0.204
0.6	0.209	0.215	0.220	0.225	0.231	0.236	0.241	0.246	0.251	0.256
0.7	0.261	0.266	0.271	0.275	0.280	0.284	0.289	0.293	0.297	0.301
0.8	0.305	0.308	0.312	0.315	0.318	0.321	0.324	0.326	0.329	0.331
0.9	0.332	0.334	0.335	0.335	0.335	0.335	0.334	0.332	0.329	0.325
1.0	0.312									

where Q = flowrate, m³/s
n = Manning coefficient of friction
D = diameter of conduit
S = slope of energy grade line, m/m.
$^b d$ = depth of flow
Note: m³/s × 35.3147 = ft³/s
m × 3.2808 = ft

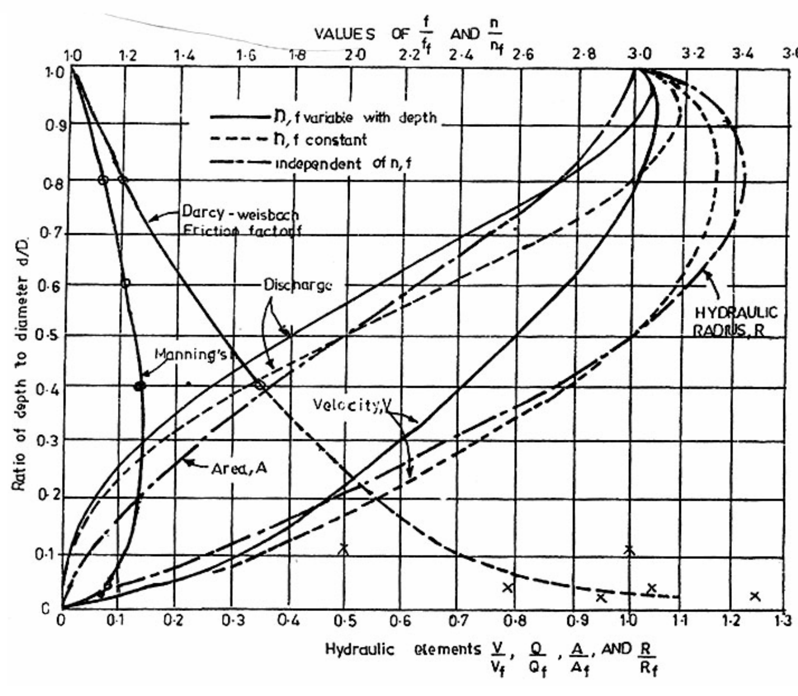

Figure 6.1: Hydraulic element graph for circular channel

(c) Design Calculations

The following color code and notations have been used for design of sewer in this document:

(i) Trunk sewer are marked in red on the drawing

(ii) Main sewer are marked in pink on the drawing

(iii) Branch sewer are marked in green on the drawing

The layout of trunk sewer has been finalized first based on the distribution of population and topography. It extends farthest from the outfall work. The manholes of the trunk sewer are marked as 0, 1, 2 etc. commencing from outfall end. The sewers joining the trunk sewer from left and right main is denoted by L and R respectively. This L & R is followed by a number denoting the manhole number of sewer. Thus R1O denotes the main sewer to the right of trunk sewer joining at manhole No 10. This figure is followed by a number which denotes the manhole number on main sewer thus R10.3 denotes manhole number 3 on right main sewer which is joining the trunk sewer at manhole number 10. Further the branch sewer is denoted by L or R (representing left and right) followed by a subscript (1, 2, 3, etc.) if number of branch sewer from some particular MH is more than one. Thus L_3 R10.3 denotes the third left branch sewer connecting MH number 3 on main sewer. This figure is further followed by a number denoting the manhole number on branch sewer thus L_3R10.3.2 denotes second MH on L_3 branch sewer. Following this notation L_4 R5.3.4 denotes 4th MH of L_4 branch sewer connecting to 3rd MH of right main sewer which in turn connecting to 5th MH on trunk sewer.

The notation followed for representing the sewer section is also similar to above except it represent section between the specified MH (explained above) to the next down stream MH. Thus L_4R5.3.4 represents a sewer section starting 4th manhole to 3rd manhole on L_4 branch sewer. The intercepting tank (septic tank) has been denoted by prefixing ST on the MH succeeding the tank.

(i) Building Sewer

Although design of building sewers is beyond the scope of this report, but design of building sewer associated with the farthest intercepting tank (STL_1R11.6.2) has been carried out. The IL of outlet of this particular septic tank being farthest from the disposal system represents the critical design parameter and has been taken as reference level. In present case maximum length of building sewer has been considered as 80 m and intercepting tanks have been located accordingly. Representative sewer design calculation is presented as under.

A. Inflow in the septic tank

Design population for the septic tank = 200,

Quantity of water supplied = 200 l/c.d

Length of sewer from first manhole = 80m (septic tanks have been located in such a manner that this is the maximum distance)

Assuming 80% of water supply as wastewater

total discharge reaching to septic tank = 200 X 0.8 X 200 = 32000 l/d

Peak flow (assuming the peak factor 3) in l/s = $\frac{32000x3}{24x60x60} =$ 1.12 l/s

It is assumed that first 20 m of sewer receives 25% of this quantity, next 20 m receives 50% and the balance stretch receives 100%.

B. First 20 m stretch:

Discharge Q = 0.28 l/s, Coefficient of friction n = 0.013

Assume diameter of pipe as 150 mm and slope 1 in 60 (both these are recommended minimum)

$$k' = \frac{0.013\,x 0.28\,x 10^{-3}}{(0.15)^{2.67}\,x (0.0167)^{0.5}} = 0.0044$$

From Table 6.1 for this of k' value, d/D value is 0.083 and a/A (ratio of area of flow to area when pipe is running full) is 0.03

Hence velocity of actual flow in pipe

$$\mathrm{v} = \frac{0.28x10^{-3}}{0.03xpipearea} = \frac{0.28x10^{-3}}{0.03x17.67x10^{-3}}$$

$$= 0.53 < 0.6\text{m/s}$$

Since self cleansing is not possible a higher slope value of 1 in 50 adopted.

For this slope the values for $k' = 0.00407$, $d/D = 0.08$, $a/A = 0.025$ and $v = 0.63 > 0.6$ m/s (Calculation is same as shown above)

Hence provide 150 mm diameter pipe at a slope of 1 in 50

C. Next 20 m stretch

$Q = 0.56 \times 10^{-3}$ m^3/s, Assume 150 mm diameter and slope of 1 in 60

The values of $k' = 0.0089$, $d/D = 0.116$, $a/A = 0.05$ and $v = 0.64 > 0.6$ m/s

D. Next 40 m stretch

$Q = 1.12 \times 10^{-3}$ m^3/s, Assume 150 mm diameter and slope of 1 in 60

The values of $k' = 0.0178$, $d/D = 0.161$, $a/A = 0.1$ and $v = 0.64 > 0.6$ m/s

E. Invert Level for the Farthest Septic Tank

Assuming ground level as 100 m datum and depth at first manhole (sewer connection manhole) as 0.25 m IL of inlet to the septic tank is:

$$\text{IL of inlet} = 100 - \left[\frac{20}{50} + \frac{20}{60} + \frac{40}{60}\right] - 0.25 = 98.35 \text{ m}$$

IL of outlet of septic tank is taken 0.05 m below IL of the inlet (IS: 2470- Part I, 1985).

$$\text{IL of outlet} = 98.35 - 0.05 = 98.3 \text{ m}$$

(ii) Small Bore Sewers

The design of sewers being repetitive can be summarized in tabular form and shown in Table 6.2 on subsequent pages.

Table 6.2: Computation table for design of small bore sewers

Sl. No.	Name of Sewer	Present population (Nos)	Total Population (including future)	Total Discharge (l/s)	Design Discharge (l/s)	Length (m)	Size of sewer (mm)	slope (1 in)	V_{full} (m/s)	V_{actual} (m/s)	U/E level (m)	L/E level (m)
1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**
1	L_1R11.6.1&2	200	200	0.37	0.74	140	100	300	0.385	0.31	98.3	97.83
2	R_1R11.6.1&2	160	200	0.37	0.74	70	100	300	0.385	0.31	98.3	98.06
3	R11.6	360	400	0.74	1.48	50	100	500	0.3	0.3	98.73	97.73
4	R_1R11.5.1	10	170	0.315	0.63	160	100	280	0.398	0.32	98.3	97.73
5	R11.5	370	570	0.855	1.71	60	100	600	0.272	0.304	97.73	97.63
6	R_1R11.4.1	240	240	0.444	0.888	20	100	200	0.471	0.36	98.3	98.2
7	R11.4	610	810	1.299	2.698	32	150	800	0.31	0.3	97.63	97.59
8	L_1R11.3.1	240	240	0.444	0.888	46	100	200	0.471	0.35	98.3	98.07
9	R11.3	850	1050	1.743	3.486	50	150	800	0.31	0.315	97.59	97.53
10	R_1R11.2.1	240	240	0.444	0.888	116	100	200	0.471	0.35	98.3	97.72
11	R11.2	1090	1290	2.187	4.374	40	150	900	0.292	0.32	97.53	97.48
12	R_1R11.1.1	320	320	0.592	1.185	20	100	100	0.667	0.54	98.3	98.1
13	R11.1	1410	1610	2.779	5.559	38	200	1200	0.31	0.305	97.48	97.45
14	10 & 11	1410	1610	2.779	5.559	176	200	1200	0.31	0.305	97.45	97.3
15	9	1410	1710	3.167	6.334	80	200	1200	0.31	0.323	97.3	97.23

Table 6.2: Contd …….

1	2	3	4	5	6	7	8	9	10	11	12	13
16	8	1410	1960	3.62	7.24	80	200	1200	0.31	0.33	97.23	97.16
17	7	1450	2000	3.703	7.406	70	200	1200	0.31	0.323	97.16	97.1
18	R7.1	40	40	0.074	0.148	34	100	100	0.667	0.314	98.3	97.96
19	R6.1	16	16	0.03	0.06	20	100	50	0.943	0.38	98.3	97.9
20	4,5 & 6	1466	2016	3.734	7.468	144	200	1200	0.31	0.33	97.1	96.98
21	RLR2.9.2& LR2.9.2	160	160	0.296	0.592	230	100	300	0.385	0.301	98.45	97.683
22	RLR2.9.1	6	6	0.011	0.022	50	100	50	0.943	0.25	98.3	97.8
23	LR2.9.1	166	166	0.307	0.615	44	100	300	0.385	0.313	97.68	97.54
24	RR2.9.1	64	64	0.12	0.24	80	100	150	0.545	0.305	98.3	97.77
25	R2.9	230	230	0.426	0.852	60	100	400	0.334	0.3	97.54	97.39
26	LR2.8.1	85	85	0.157	0.314	15	100	150	0.545	0.36	98.8	98.2
27	R2.8	315	315	0.584	1.168	44	100	400	0.334	0.32	97.39	97.28
28	LR2.7.5 LR2.7.4	155	215	0.398	0.796	120	100	250	0.422	0.34	98.3	97.82
29	LLR2.7.3.2	38	38	0.07	0.14	36	100	100	0.667	0.32	98.3	97.94
30	LLR2.7.3.1	50	278	0.515	1.03	35	100	250	0.422	0.37	97.94	97.8

Table 6.2: Contd …….

1	2	3	4	5	6	7	8	9	10	11	12	13
31	LR2.7.3	205	493	0.913	1.826	90	100	400	0.334	0.357	97.8	97.58
32	RLR2.7.2	0	320	0.592	1.184	50	100	300	0.385	0.34	98.3	98.14
33	LR2.7.2	205	813	1.505	3.01	125	150	500	0.392	0.35	97.58	97.33
34	LLR2.7.1	20	20	0.037	0.074	20	100	100	0.54	0.36	98.3	98.1
35	LR2.7.1	225	833	1.542	3.084	52	150	500	0.392	0.352	97.33	97.23
36	R2.7	540	1148	2.126	4.252	125	150	800	0.31	0.334	97.23	97.07
37	RR2.6.1	18	18	0.034	0..068	30	100	100	0.667	0.365	98.3	98
38	R2.6	558	1166	2.159	4.318	50	150	800	0.31	0.34	97.07	97.01
39	LR2.5.3	40	136	0.252	0.504	34	100	200	0.471	0.34	98.3	98.13
40	LR2.5.2	40	136	0.252	0.504	140	100	200	0.471	0.34	98.13	97.43
41	LLR2.5.1	56	96	0.178	0.356	60	100	150	0.545	0.33	98.3	97.9
42	RLR2.5.2	77	173	0.32	0.64	15	100	150	0.545	0.41	98.3	98.2
43	RLR2.5.1	77	173	0.32	0.64	48	100	150	0.545	0.41	98.2	97.88
44	LR2.5.1	117	313	0.579	1.158	125	100	300	0.385	0.35	97.43	97.01
45	R2.5	675	1479	2.74	5.48	80	150	800	0.31	0.34	97.01	96.91

Table 6.2: Contd …….

1	2	3	4	5	6	7	8	9	10	11	12	13
46	RR2.4.4	240	240	0.445	0.89	34	100	200	0.471	0.36	98.3	98.13
47	RR2.4.3	155	155	0.287	0.574	50	100	200	0.471	0.325	98.3	98.05
48	RR2.4.2	395	595	1.102	2.204	80	100	300	0.385	0.36	97.63	97.36
49	RR2.4.1	395	595	1.102	2.204	94	100	300	0.385	0.36	97.36	97.05
50	R2.4	1070	2073	3.84	7.68	12	200	1100	0.32	0.345	96.91	96.9
51	LR2.3.1	56	56	0.104	0.208	100	100	150	0.545	0.33	98.3	97.63
52	R2.3&2.2	1126	2129	3.943	7.886	160	200	1250	0.3	0.33	96.9	96.77
53	LR2.1.1	16	16	0.0296	0.0592	80	100	100	0.667	0.35	98.3	97.5
54	R2.1	1142	2145	3.972	7.944	57	200	1250	0.3	0.328	96.77	96.72
55	L3.1	40	40	0.074	0.148	15	100	100	0.667	0.314	98.3	98.15
56	3	1506	2056	3.807	7.614	112	200	1250	0.3	0.327	96.98	96.89
57	R_1L2.3.2&1	350*+200**	350*+200**	0.636	1.272	260	100	300	0.385	0.37	98.3	97.43
58	RL_1L2.3.2	100*	100*	0.116	0.232	20	100	180	0.497	0.305	98.3	98.18
59	L_1L2.3.2	100*	100*	0.116	0.232	76	100	180	0.497	0.305	98.18	97.76
60	RL_1L2.3.1	150*+200**	150*+200**	0.405	0.81	20	100	300	0.385	0.33	98.3	98.23

Table 6.2: Contd …….

1	2	3	4	5	6	7	8	9	10	11	12	13
61	L_1L2.3.1	250*+200**	250*+200**	0.521	1.042	154	100	400	0.334	0.31	97.76	97.37
62	L2.3	600*+400**	600*+400**	1.157	2.314	110	100	500	0.3	0.33	97.37	97.15
63	R_1L2.2.1	40	140	0.259	0.518	120	100	200	0.471	0.33	98.3	97.7
64	L2.2	600*+400**+40	600*+400**+ 140	1.416	2.832	42	150	600	0.358	0.34	97.15	97.08
65	L_1L2.1.1	100*+300**	100*+300**	0.463	0.926	160	100	200	0.471	0.36	98.3	97.5
66	L2.1	700*+700**+40	700*+700**+ 140	1.879	3.758	40	150	900	0.32	0.31	97.08	97.03
67	2	700*+700** +2688	700*+700**+ 4341	9.659	19.318	80	300	1200	0.4	0.43	96.72	96.65
68	L1	20	20	0.037	0.074	10	100	100	0.667	0.38	98.3	98.2
69	1	700*+700** +2708	700*+700**+ 4361	9.696	19.392	90	300	1200	0.4	0.435	96.65	96.58

Note: 1. * and ** denotes the population using community kitchen and toilets.

2. Representative calculations are given in sub sequent pages

A. <u>Design of Branch Sewer</u>

Sewer section L_1R11.6.1 and L_1R11.6.2 (serial no.1 of design table)

Total length of sewer section = 140 m, Design population = 200

Average wastewater flow rate in l/s = population x 80% of water supply

$$= \frac{200x0.8x200}{24x60x60} = 0.37 \text{ l/s}$$

Design flow (assuming peak factor as 2) = 2 x 0.37 = 0.74 l/s

Since flow is less assume 100 mm diameter of pipe and a slope of 1 in 300

$$k' = \frac{nQ}{D^{8/3} S^{1/2}} = \frac{0.013x0.74x10^{-3}}{0.1^{8/3}\, x\left(1/300\right)^{1/2}} = 0.0864$$

d/D value for this value of k' (from Table 4.3) = 0.36

From the hydraulic relation Figure for above d/D value, a/A = 0.31

Actual velocity in pipe $v = \dfrac{0.74x10^{-3}}{0.31x\left(\dfrac{\pi}{4}x0.1^2\right)}$ = 0.36 > 0.3 m/s

Hence assumed diameter and slope is acceptable.

Velocity when the pipe runs full (Column 10 of design sheet)

$$V_{full} = \frac{0.397}{n}x(D)^{2/3}x(S)^{1/2} = \frac{0.397}{0.013}x(0.1)^{2/3}x\left(\frac{1}{300}\right)^{1/2}$$

= 0.376 < 3 m/s Hence OK

IL of lower end = 98.3 m (i.e. IL of outlet of septic tank),

IL of upper end = 98.3 -140 x (1/300) = 97.83 m

B. <u>Design Of Main Sewer</u>

Sewer section R11.6 (serial no. 3 of design sheet)

Length of sewer section = 50 m, Design population = 400

Design flow (assuming peak flow factor as 2) in l/s = 1.48 l/s

Assuming pipe diameter as 100 mm and slope of 1 in 600

$$k' = \frac{nQ}{D^{8/3} S^{1/2}} = \frac{0.013x1.48x10^{-3}}{0.1^{8/3}\, x\left(1/600\right)^{1/2}} = 0.220$$

d/D value for this value of k' (from Table 4.3) = 0.62

From the hydraulic relation figure for above d/D value, a/A = 0.65

Actual velocity in pipe $v = \dfrac{1.48x10^{-3}}{0.65x\left(\dfrac{\pi}{4}x0.1^2\right)} = 0.289 < 0.3$ m/s

Hence assumed diameter and slope is not acceptable.

Assume slope of 1 in 500

k' = 0.201, d/D = 0.584, a/A = 0.62 and v = 0.30 m/s

Hence provide 100 mm diameter pipe with slope of 1 in 500.

Velocity when the pipe runs full (Column 10 of design sheet)

$$V_{full} = \frac{0.397}{n}x(D)^{2/3}x(S)^{1/2} = \frac{0.397}{0.013}x(0.1)^{2/3}x\left(\frac{1}{500}\right)^{1/2}$$

= 0.292 < 3 m/s Hence OK

IL of lower end = 97.83 m, IL of upper end = 97.83 -50 x (1/500) = 97.73 m

C <u>Design of Trunk Sewer</u>

Sewer section 10 and 11

Total length of section = 176 m, Design population =1610, Design flow in l/s = 5.559 l/s

Assume a diameter of 150 mm and slope of 1 in 900

$$k' = \frac{nQ}{D^{8/3}S^{1/2}} = \frac{0.013x5.559x10^{-3}}{0.15^{8/3}x\left(1/900\right)^{1/2}} = 0.343$$

from the table it can be seen that this value of k' is not feasible

Hence assume slope of 1 in 800 and diameter of 150 mm

$$k' = \frac{nQ}{D^{8/3}S^{1/2}} = \frac{0.013x5.559x10^{-3}}{0.15^{8/3}x\left(1/800\right)^{1/2}} = 0.324$$

d/D value for this value of k' (from Table 4.3) = 0.99

From the hydraulic relation figure for above d/D value, a/A $\simeq$ 1.0

actual velocity in pipe $v = \dfrac{5.559x10^{-3}}{1x\left(\dfrac{\pi}{4}x0.15^2\right)} = 0.315 > 0.3$ m/s

although assumed diameter and slope is acceptable to avoid the depth of excavation

Assume slope of 1 in 900 and diameter of pipe as 200 mm k’ = 0.159, d/D = 0.515, a/A = 0.5,

$$v = \frac{5.559x10^{-3}}{0.5x\left(\frac{\pi}{4}x0.2^2\right)} = 0.34 \text{ m/s}$$

Assume a slope of 1 in 1200

k’ = 0.184, d/D = 0.55, a/A = 0.58, v = 0.305 > 0.3

Hence provide 200 mm diameter pipe with slope of 1 in 1200.

velocity when the pipe runs full (Column 4 of design sheet)

$$V_{full} = \frac{0.397}{n}x(D)^{2/3}x(S)^{1/2} = \frac{0.397}{0.013}x(0.2)^{2/3}x\left(\frac{1}{1200}\right)^{1/2} = 0.31 < 3 \text{ m/s}$$

Hence OK

IL of lower end = 97.45 m, IL of upper end = 97.45 - 176 x (1/1200) = 97.30 m

D. Sewer appurtenances:

Clean outs are used instead of manholes in small bore system except, at major junctions and should be located at all upstream ends, intersections of sewer lines, major changes in direction, at high points and at at intervals of 60-100 m in a straight reaches to long flat sections. Ventilation is not necessary for small bore system if they are laid on a falling gradient. A vent cleanout to release air may be provided at every hump (CPHEEO, 1993).

6.3 Intercepting Tanks (Septic Tank)

Intercepting tanks in present case is a combined sedimentation and digestion tank (septic tanks) where the sewage is held for one to two days. During this period the suspended solids settle down to the bottom and digested anaerobically thereby causing reasonable reduction in volume of sludge. For efficient removal of suspended solids, the septic tank should be of sufficient capacity with proper inlet and outlet. The septic tank may be designed for 1 to 2 days of wastewater retention. The IL of outlet of septic tank should be 50 mm below the IL of inlet pipe. Since design of intercepting tank is beyond scope, of this design detailed design of this has not been given (IS: 2470-Part-I, 1985, can be referred to for designing of septic tanks).

In this design the sizes of septic tanks have been taken as recommended in the IS: 2470-Part-I, 1985 and are given in Table 6.3 and 6.4, however if required the septic tank can be designed as per the criteria given in the above mentioned IS Code. Typical drawings of septic tanks applicable to present case are shown in Figure 6.2 and 6.3.

Table 6.3: Recommended size of septic tanks up to 20 users

No. of users	Length in m	Breadth in m	Liquid depth (cleaning interval of)	
			2 years	3 Years
5	1.5	0.75	1.0	1.05
10	2.0	0.9	1.0	1.40
15	2.0	0.9	1.3	2.0
20	2.3	1.10	1.3	1.8

Note 1: The capacities are recommended on the assumption that discharge from only WC will be treated in the septic tank.

Note 2: A provision of 300 mm should be made for free board.

Note 3: The sizes of septic tank are based on certain assumption on peak discharges as estimated this IS and while choosing the size of septic tank exact calculations shall be made.

(Source: IS: 2470- Part 1, 1985)

Table 6.4: Recommended size of septic tanks for residential colonies

No. of users	Length in m	Breadth in m	Liquid depth (cleaning interval of)	
			2 years	3 Years
50	5	2.0	1.0	1.24
100	7.5	2.65	1.0	1.24
150	10.0	3.0	1.0	1.24
200	12.0	3.30	1.0	1.24
300	15.0	4.0	1.0	1.24

Note 1: For population over 100 the tank may be divided into two independent chambers for maintenance and cleaning

Note 2: A provision of 300 mm should be made for free board.

Note 3: The sizes of septic tank are based on certain assumption on peak discharges as estimated this IS and while choosing the size of septic tank exact calculations shall be made.

(Source: IS: 2470- Part I, 1985)

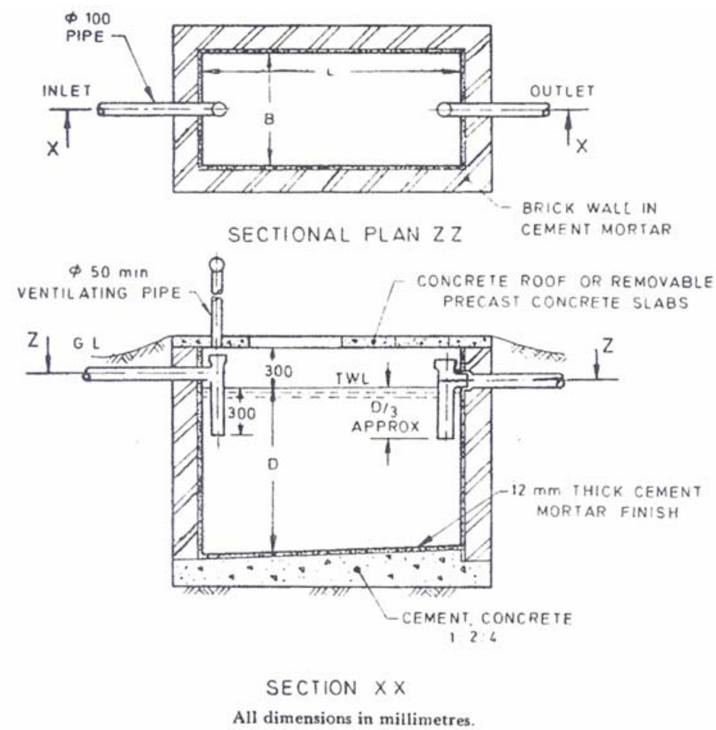

Figure 6.2: Typical sketch of single compartment septic tank upto 20 users
(Source: IS: 2470- Part I, 1985)

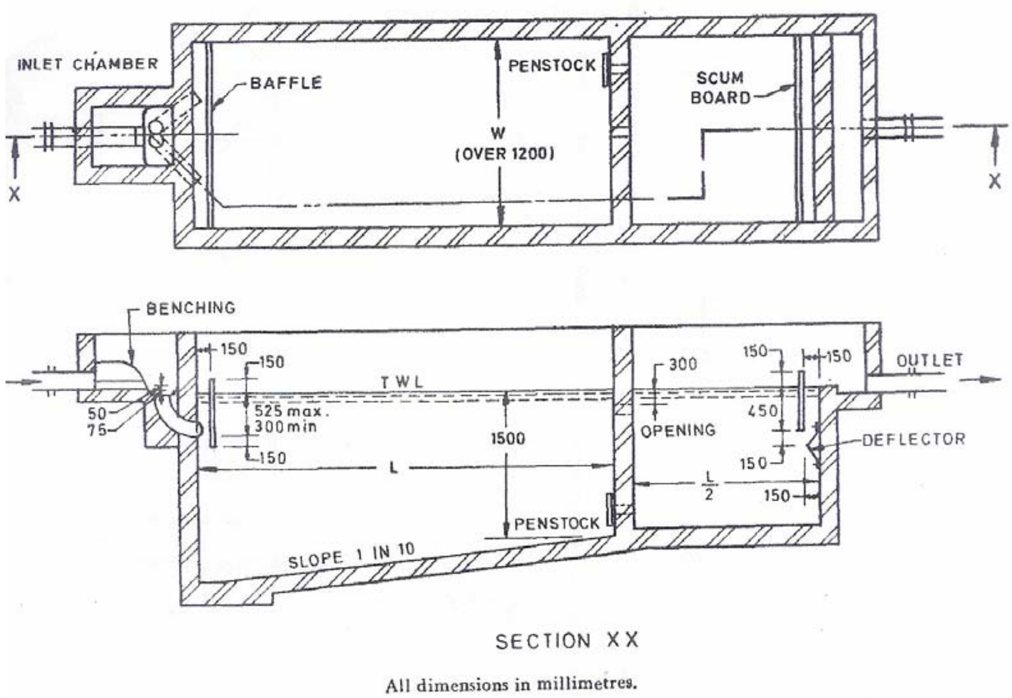

Figure 6.2: Typical sketch of single compartment septic tank upto 20 users
(Source: IS: 2470- Part I, 1985)

6.4 Salient Features of Designed Sewer System

- Design velocities have been kept as ≥ 0.3 m/s to have proper functioning with least maintenance problems
- In cases where future expansions are likely slopes has been worked out in such a way that they cater to both present population as well as future expansion.
- IL of the farthest septic tank provides sufficient factor of safety from houses to sewer system and slopes steeper than 1: 60 can be provided in most of the cases.
- Slope of the main trunk sewer has been adjusted in such a fashion that it matches with the IL of the manhole where it is being connected and the velocities are approximately same (purpose has been to eliminate the need for dissipating energy).
- The sewer diameter and slopes has been designed in such a fashion that it optimizes the cost of construction by reducing the excavation depth.
- The minimum velocity in the sewer has been considered as critical design criteria rather than sewer flowing full. If latter is considered as critical criteria it will lead to impractically flatter slopes and much lesser than the designed velocity.

CHAPTER 7

DESIGN OF SEWAGE TREATMENT PLANT AND DISPOSAL SYSTEM

7.1 Introduction

Clarified sewage conveyed through the sewerage system is collected in a sump in the vicinity of sewage treatment plant. Normally pumps are used for passing the sewage through the STP. In the STP clarified sewage is given secondary treatment in order to meet the discharge standards to surface water bodies. Two alternate technologies for secondary treatment are considered in this design. Process flow scheme of the STP considered is as shown in Figure 5.1.

The treated effluent is temporarily held in a fishpond and from there pumped and used for irrigation of golf course and plantation within the cantonment. Excess water is allowed to overflow the fish pond into road side drain.

7.2 Design of STP

7.2.1 Design of Sump and Pump

(a) Design Considerations

(i) Desirable retention time in the sump is 10-30 minutes with pumping cycle (start of pump to stop of pump) of minimum 15 minutes (Metcalf and Eddy, Inc., 1983). In the present case, the capacity of the sump has been calculated based on peak hours of sewage inflow and sewage pumping schedule through the STP. It is proposed that the sewage will be on an average pumped for 6 hours in a day and three times a day at 6.00 AM, 1.00 AM and 6.00 PM.

(ii) Since the maximum inflow is going to be in the morning it is assumed that 60% of daily average inflow is in the morning hours from 5.30 AM to 8.30 AM i.e. 3 hours, out of balance 40% discharge it is assumed that 25% is received in the evening hours between 6.00 PM to 8.00 PM and rest 15 % is during remaining period. Considering the morning hours (critical conditions) inflow is to be pumped out in 3 hours, hence sump capacity has been considered as 20% of daily average flow.

(iii) Working depth of the sump has been considered as 2.5 m.

(iv) For pumping the sewage through the STP, three pumps of equal capacity (two in operation & one as standby) have been provided. For taking care of power failures a DG set with sufficient capacity to run one pump is to be provided.

(v) The head of pump has been calculated taking IL of overflow weir (provided in the fish pond) as reference and then working out the head backward at each stage. Due to lack of exact data head losses have been assumed to be 0.75m (includes friction losses and other minor losses).

(vi) Pumping operation (starting and stopping) has been considered to be automated (using the level sensors) with a provision to operate it manually also. In view of this capacity of sump assumed may also not be fully utilized but considering the manual operation and factor of safety sump capacity is considered as 20% of daily average inflow.

(b) Design calculations

(i) Sump

The average flow reaching to the sump = 9.696 l/s = 837.73 m^3/d

Since the wastewater flow peaks in morning and evening hours and partially during mid day, it is proposed to use a total of 6 hours pumping, three times a day 5.30 AM, 12.00 at noon and 6.00 PM.

The capacity of sump required = 837.73 X 0.2 = 167.55 m^3

Assuming a working depth of 2.5 m

Surface area required for sump = 67.02 m^2

It is proposed to construct a circular sump,

The diameter of sump = $\sqrt{\dfrac{67.02 X 4}{\pi}}$ = 9.23 m, assume a diameter of 9.25 m

The IL of the sump = 96.58 - 2.5 = 94.08 m

Provide a sump of 9.25 m diameter and 5.9 m total depth with a working depth of 2.5 m. The bottom of the sump shall be provided with the slope towards pump suction inlet which is a small pit having bed level below the sump bottom level so that solids settling at bottom are drawn towards pump inlet due to hydraulic and gravitational forces, thereby providing self cleansing to sump bottom. Pump can be located over the sump by providing RCC cover at the IL of 98.58 m (thereby providing a free board of 2 m which provide for additional storage capacity in case of failures). This cover has been provided with suitable opening and ladder for maintenance. The working depth of is to be partitioned with a provision of inter connection for ease in maintenance.

(ii) Pump

A. <u>Capacity</u>

Assuming 6 hours pumping duration, capacity of pump required (in lpm)

$$= \frac{837.73 X 1000}{6 X 60} = \ 2327 \ \text{say } 2400 \text{ lpm}$$

It is proposed to operate 2 pumps simultaneously, hence capacity of one pump

$$= \frac{2327}{2} = 1163 \text{ lpm}$$

B. <u>Head calculations</u>

The STP using the VSB is the critical design consideration, assuming IL of overflow drain in fish pond as reference and 0.25 m as head required for flow from fish pond to storm water drain head required at fish pond

= 0.25 m

Head required at VSB

= water depth in VSB + head loss in VSB bed + Hydraulic head at inlet of VSB

= 0.4 + 0.1 + 0.2 = 0.7 m

hence total head required at VSB = 0.25 + 0.7 = 0.95 m

IL at VSB inlet assuming ground level as reference =100.95 m

Hence static head required for the pump = 100.95 – 94.08 = 6.87 m

Since the pump inlet is to be provided in a suction pit, assuming depth of suction pit as 0.4 m total head required

$= 6.87 + 0.4 = 6.91$ m

Provide three pumps (two in operation and one stand by) of 1200 lpm and 6.91m head. While selecting the pump it should be considered that this is to be used for clarified liquid.

7.2.2 Design of STP

To design a treatment plant proper characterization of wastewater is one of the most important step. Since characteristics of sewage vary from place to place, it has been decided to use the characteristics based on the actual sample analysis reports of four different cantonments. The data is tabulated in Table 7.1.

Table 7.1: Characteristics of raw sewage (all values in mg/l)

Parameter	Cantt A	Cantt B	Cantt C	Cantt D	Design concentration
BOD	112	125	118	125	132.54
COD	360.2	278	368	379.85	405.9
TDS	492	450	630	1160	1336.16
TSS	155	95	218	370	445.89
pH	7.7	6.5	7.1	4.5	-
Sulphates	-	50.20	--	-	50.20
Chlorides	-	20.16	-	-	22.16
TN	-	-	-	-	
TP	-	-	-	-	

(Source: Analysis reports)

The design characteristics considered has been given in column 6 of Table 7.1 and has been calculated using the following formula considering the mean and standard deviation:

Design Parameter = Mean value of Parameter + 2 (Standard deviation)

Septic tanks/intercepting tanks are assumed to remove BOD and TSS by 40% and 50% respectively. As per the literature BOD and TSS are removed by 60% and 50-70% (Salvato, 1982) respectively.

(a) Representative Calculation of Design Concentration (Column 6 Of Table 7.1):

$$\text{Mean of BOD} = \frac{112 + 125 + 118 + 125}{4} = 120$$

Standard deviation

$$= \sqrt{\frac{\sum_{i=1}^{n}(x_i - 120)^2}{(n-1)}}$$

$$= \sqrt{\frac{(112-120)^2 + (125-120)^2 + (125-120)^2 + (118-120)^2}{(4-1)}} = 6.27$$

Hence design concentration = 120 + 2 x (6.27) = 132.54

(ii) <u>Clarified sewage</u>

The pollutant removal in Septic Tank can be taken as given below:

BOD	60%	
Suspended solids	50-70%	(Salvato, 1982)

Assuming poor maintenance, resuspension and turbulence BOD and SS removal has been considered in design as 40% and 50% respectively. The effluent that has to be handled by the STP will thus be having the following characteristics.

Design BOD = 0.60x 132.54 = 79.52 mg/l, say 80 mg/l

Design SS = 0.5x405.9 = 202.95 mg/l

BOD loading per day = 837.73 x 1000 x 80= 67018400 mg/d = 67.02 kg/d

The general standards of discharge of environment pollutants in inland surface water and land disposal are given in Table 7.2. From this and influent pollutant concentration it can be inferred that if the treated effluent is to be discharged on land no secondary treatment is required but since the overflow is to be discharged in surface inland body the secondary treatment is to be given before disposal.

Table 7.2: General standards of discharge of environment pollutants

Parameters	Standards	
	Inland surface water	Land of irrigation
Suspended solids mg/l	100	200
pH value	5.5 -9	5.5 -9
BOD	30	100
COD	250	-

(Source: Environment (Protection) Rules, 1986)

7.2.2.1 Design of Activated Sludge Plant

(a) Design Considerations

(i) Activated sludge plant with the scheme shown in Figure 4.1 is considered in the present case. Surface aerators have been proposed to be used the aeration tank thus will be having a completely mixed condition.

(ii) Due importance has been given to sludge volume index (SVI) to ensure better liquid solid separation in secondary clarifier. The SVI of 120 has been considered in the design the desired range of SVI is 80–150 (CPHEEO, 1993). Although zone settling is normally assumed to take place in secondary clarifier, but in the present case due to low strength sewage (municipal sewage) the MLSS concentration in the aeration tank can not be maintained at desired level (2000 - 4000 mg /l) and due to this apart from zone settling flocculant settling will also take place and hence additional depth of secondary clarifier has been considered.

(iii) In the design of activated sludge plant the primary variable have been assumed are sludge retention time and hydraulic retention time. Using these variables other parameters have been checked/calculated.

(iv) It is proposed to waste the excess sludge from secondary clarifier. Since influent BOD is less and some MLSS will be discharged with the effluent (40 mg/l is considered for this design) desired MLSS concentration in the aeration tank is very difficult to maintain and requires continuous recycling of sludge. But considering various practical aspects this is not feasible, hence recycling pump with auto start and stop with desired rest period (5-10 minutes as per manufacturer's instruction) are to be provided and near continuous recycling is to be ensured. Further to this since sewage inflow is not uniform and thus, shock loading to ASP can not be avoided, which makes the task of minimum desired MLSS concentration in aeration tank further complicated. Since the MLSS concentration and thus quantity of wastage sludge being small it is considered to wastage the sludge once in a day only if required.

(v) Secondary clarifier has been designed taking into consideration overflow rate, solid loading rate and weir loading rate. Current practice favors a minimum side water depth of 3.5 meters. Deeper tanks provide greater flexibility of operation and larger margin of safety, hence additional depth for the tank has been considered which takes care for flocculant settling also.

(vi) Since municipal sewage is rich in nutrients, there is no requirement of adding this and hence in this design it has not been taken into consideration.

(b) Design Equation and Criteria

(i) Activated sludge plant

Hydraulic retention time (HRT) $t = \frac{V}{Q}$

Where t is hydraulic retention time in days, V is volume in m^3, and Q is influent flow rate in m^3/d.

Sludge retention time (SRT) $\Theta_c = \frac{y(S_0 - S_e)}{tX} - k_d$

Where Θ_c is SRT in days, y is maximum yield coefficient i.e. ratio of microbial mass synthesized to the mass of substrate utilized, k_d is endogenous respiration rate constant in day^{-1}, S_0 and S_e are influent and effluent organic matter concentration, conventionally measured as BOD_5, X is mass concentration in aeration tank in mg/l. Solving above equation for X and replacing t as V/Q the equation can be written as:

$$VX = \frac{\Theta_c y Q(S_0 - S_e)}{1 + k_d \Theta_c}$$

Food to micro-organism ratio can be written as:

$$\frac{F}{M} = \frac{Q(S_0 - S_e)}{VX}$$

Sludge wastage rate is given by:

$$Q_w X_s + Q_e X_e = yQ\ (S_0 - S_e) - k_d VX + QX_i$$

Where Q_e is effluent flow rate, X_e, and X_i is MLSS concentration in effluent and in influent (taken as zero in this design) respectively and X_s is MLSS concentration in waste activated sludge from secondary settling tank underflow in mg/l and is taken as:

$$X_s = \frac{10^6}{SVI}$$

Effluent flow rate is given by

$$Q_e = Q - \frac{yQ(S_0 - S_e) - k_d VX + QX_i - QX_e}{X_s - X_e}$$

Where SVI is sludge volume index, X_e is concentration of MLSS in effluent

Sludge recycle ratio is given by:

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

Where Q_r is sludge recirculation rate.

The total oxygen requirement of the process may be formulated as follows:

$$\text{Oxygen required (g/d)} = \frac{Q(S_0 - S_e)}{f} - 1.42(Q_w X_s + Q_e X_e)$$

Where f is the ratio of BOD_5 to ultimate BOD and 1.42 is oxygen demand of biomass in g/g.

Aeration equipment shall be designed with a peaking factor of 1.5-2 times the average BOD load. Two speed aerators are desirable to provide operating flexibility to cover a wide range of O_2 demand condition.

The BOD removal in the ASP is a function of SRT Figure 7.1 shows the SRT and temperature relationship for 90-95% removal BOD.

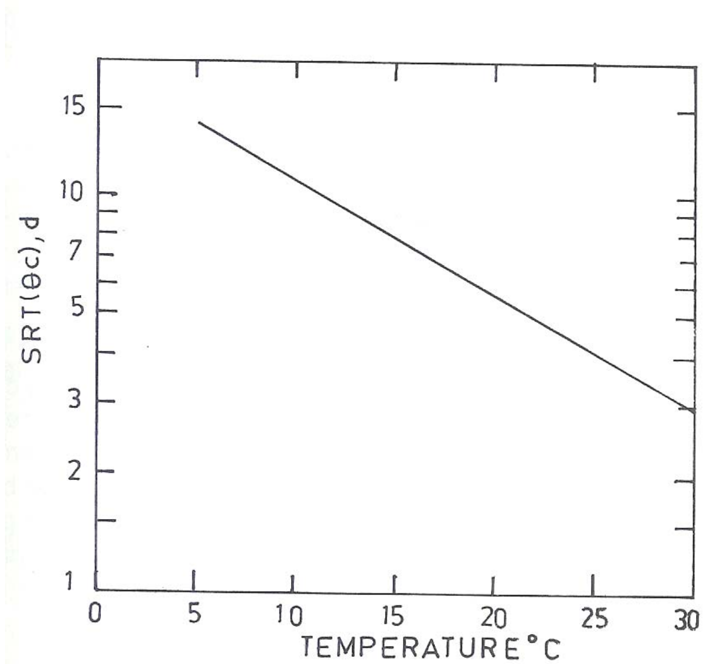

Figure 7.1: SRT as a function of aeration temperature for 90-95% removal of BOD

Design criteria applicable to ASP for municipal sewage are given in Table 7.3.

Table 7.3: Design criteria for activated sludge plant for completely mixed reactor

S No	MLSS mg/l	MLVSS / MLSS	F/M (kg BOD_5/ kg MLSS)	HRT H	Θ_c in days	Recycle Ratio (Q_r/ Q)	Kg of O_2 supplied/ kg of BOD removed	SVI in ml/g	Xs (MLSS in return sludge in mg /l)	Volumetric loading kg BOD/m^3.d.	Efficiency %
1	3000-4000	0.8	0.3-0.5	4-5	5-8	0.25-0.8	0.8-1.0	-	< 10000	-	85-92
2	1500-4000	0.2-0.4	0.3-0.5	3-15	3-15	0.25-1.0	0.9-1.3	80-150	4000-12000	0.3-1.6	

Source: Sl No. 1 (CPHEEO, 1993), Sl No.2 (Metcalf and Eddy, Inc., 2003)

(ii) Secondary clarifier

For this design circular tank with centre feed system is considered. Common diameter range for the tank is 3 to 60 m (Metcalf and Eddy, Inc., 2003). The surface area of the clarifier is calculated based on surface and solid loading rates considering average flow rate and this design is checked for peak flows. Tank inlets should be provided to dissipate influent energy and distribute flow evenly in all directions. In circular centre feed tank a common design is to use small, solid skirted cylindrical baffles.

In small tanks water location does not affect the performance of secondary clarifier significantly. In this design perimeter weir is provided. The bottom of tank is designed as hopper with gentle slope towards centre. The hopper volume shall be excluded from effective sedimentation volume of the tank. It is preferable to remove sludge continuously from secondary clarifier for ASP to avoid septicity and rising of sludge problems.

$$\text{Overflow rate} \quad = \frac{Flowrate}{surfacearea}$$

$$\text{Solid loading rate} \; = \frac{totalsolidsapplied}{surfacearea}$$

$$\text{Weir loading rate} \; = \frac{Flowrate}{perimeterclarifier}$$

Design criteria applicable to secondary clarifier for ASP is given in Table 7.4.

Table 7.4: Design criteria for secondary clarifier for ASP

Over flow rate in $m^3/m^2.d$		Solid loading rate in $kg/m^2/d$		Depth in m	Detention time in h	Weir loading rate in $m^3/m.d$
Average	Peak	Average	Peak			
16-28	40-64	4-6	8	-	-	-
15-35	40-50	3-6	9	3.5-4.5	-	<185

Source: Sl No. 1(Metcalf and Eddy, Inc., 2003) , Sl No.2 (CPHEEO, 1993)

(c) Design Calculations

(i) Activated sludge plant

A. Field Data

Influent Q = 837.73m^3/d, BOD concentration in influent = 80 mg/l

Assuming 85% removal BOD concentration in effluent = 12 mg/l

Minimum atmospheric temperature = 6^0C, Design temperature in tank (assumed) = 15^0C

B. Data from Design Criteria

SVI = 120, y = 0.5, k_d = 0.06 and f= 0.68

From Figure 7.1 SRT for this temperature θ_c = 8 days, assume HRT as 5 hours

C. Calculations

Concentration of MLSS in tank X_a = $\frac{1}{t}\left[\frac{\theta_c y(S_0 - S_e)}{(1+k_d\theta_c}\right] = \frac{24}{5}\left[\frac{8x0.5x(80-12)}{1+0.06x8}\right]$

= 882.16 say 900 mg/l

MLSS in return sludge X_s = $\frac{10^6}{SVI}$ = 8333.33 mg/l, assume 8000 mg/l

Volume of tank = Q x t = 837.73 x 5/24 = 174.53 m^3

$Q_e \approx$ 837.73 m^3/d

Excess sludge = Q_w x X_s = y x Q x (S_o – S) - k_d x V x X - Q_e X_e

= 0.5 x 837.73 x (80-12) – 0.06 x 174.53 x 900 – 837.73 x 40

= - 14451 g/d = - 14.45 kg/d

The – sign indicates there is no sludge required to be wasted, what ever sludge is produced is either flushes away with effluent or recycled.

$$\text{Recycle ratio} = \frac{900}{8000-900} = 0.126$$

Hence Q_r = 0.126 x 837.73 = 105.55 m^3/d

Provide recirculation pump of capacity 105.55 m^3/d or 76.38 say 77 lpm

Total BOD removed per day = (80-12) x 837.73 x 1000 = 56969040 mg = 56.97 kg/d

$$\text{Oxygen required (g/d)} = \frac{Q(S_0 - S_e)}{f} - 1.42(Q_w X_s + Q_e X_e)$$

= 56713 g/d = 56.71 kg/d

kg of O_2 required/ kg of BOD removed = 56.71/56.97= 0.997, provide 1 kg of O_2 / kg of BOD removed hence oxygen required = 56.97 kg/d say 57 kg /d

(ii) Secondary clarifier:

A. Field data

Average wastewater flow = 837.73 m^3/d, MLSS concentration in wastewater = 900 mg/l,

TSS concentration in wastewater permitted = 100 mg/l

B. Data derived from design criteria

Surface loading rate = 20 m^3/m^2.d, Solid loading rate = 80 kg/m^2.d

Peak solid loading rate = 210 kg/m^2.d, Peak flow factor = 2

C. Design calculations

(i) Surface area required based on surface loading rate = $\frac{Q_{avg}}{surfaceloadingrate}$

$$= \frac{837.73}{20} = 33.51 \text{ say } 34 \text{ m}^2$$

Check for surface loading for peak flow = $\frac{837.73x2}{34}$ = 49.28 m^3/m^2.d

(permissible value is 40-50 hence OK)

(ii) Area requirement based on solid loading rate

Solids in wastewater = (900 + 102.95) x 837.73 = 840201.3 g= 840.20 kg/d

Surface area required based on solid loading rate = $\frac{Q_{avg}}{solidloadingrate}$

= 840.20/80 = 10.50 m^2

At peak flow = $\frac{837.73x2x(900+102.95)}{1000x210}$ = 8.00 m^2

Hence adopt an area of 34 m^2, diameter of tank = 6.58 m

Assuming depth of 4 m, Volume of tank = 119 m^3

Hydraulic retention time in tank = $\frac{119x24}{837.73}$ = 3.4 h > 1.5 h hence OK

Check for weir loading rate

Weir loading rate = $\frac{837.73}{\pi x 6.58}$ = 40.52 m^3/m.d <125 hence OK

It can be seen from the above design calculations of ASP that for municipal sewage having low BOD and part of MLSS going out with effluent it is practically not feasible to maintain the desired concentration in aeration tank. Further to this continuous recycling of sludge has to be done although automated recirculation system can be provided, still it is not feasible to ensure this, due to power break downs and other practical considerations. Once this biological system destabilizes it's reestablishing is difficult. Further to this since sewage inflow is not uniform, shock loading to ASP can not be avoided, which makes the task of minimum desired MLSS concentration in aeration tank further complicated. Though zone settling condition are assumed in secondary clarifier, due to low MLSS concentration flocculant settling also occurs affecting the performance of the system. Hence this system can not be considered appropriate for secondary treatment of municipal sewage, though being used widely. Constructed wetland system provides a better alternative.

7.2.2.2 Design of VSB

(a) Design Considerations

(i) The VSB system has been designed to remove the BOD to the desired level, removal of all other pollutants are coincidental and not by design.

(ii) Seeing the economic value and demand in local market. *Phagmites,* has been considered for the VSB.

(iii) The outlet of VSB is considered to take care of inflows due to rain fall.

(b) Design Equation and Criteria

Two important assumptions made in designing of VSB are:

A. hydraulic gradients can be used in place of slope and

B. The VSB has four zones as shown in Figure 7.2:

- Inlet zone
- Initial treatment zone (occupy 30% of total area)
- Final treatment zone (occupy 70% of total area)
- Outlet zone

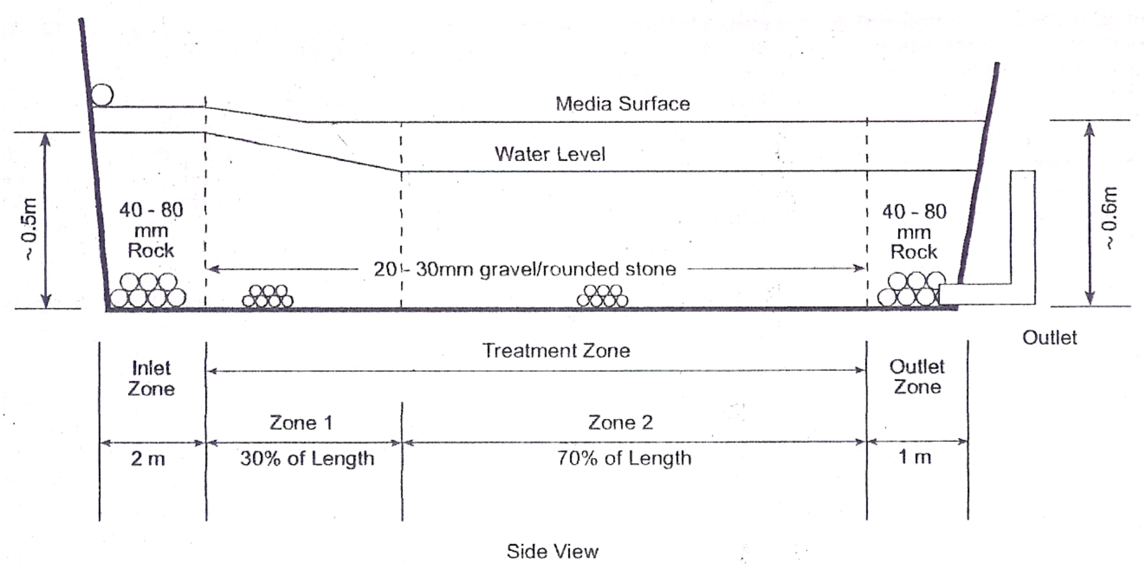

Figure 7.2: Proposed Zones in VSB

Avoiding free water surfaces has been one of the design considerations. This is considered important to control flies and mosquitoes problem. Contribution by precipitation or runoff from the catchments area must be estimated and included in the design flow, although except in very wet climate precipitation will not adversely affect the CWS performance as precipitation dilutes the sewage. However design of the outlet structure to cater for this additional flow from precipitation is considered important. Evapo-transpiration decreases hydraulic loading and thus offsets effect of precipitation.. Rainfall occurring in the form of storms of sufficient intensity and duration, can cause flooding of the wetland system. To avoid this, the collection and handling system of treated effluents, and even the outlets of individual cells, may be sized to handle both the treated effluents and the storm water.

Water level in a VSB system is controlled by the outlet elevation and hydraulic gradient. The principal aspects worked on in the design of vegetated subsurface flow constructed wetlands are hydraulic retention time, BOD and solids loading rates, size of the media and depth.

Hydraulics of the subsurface flow must be checked using Darcy's law (Crites, 1994). General form of Darcy's law that can be used for the purpose is:

$$Q = (K)(A_c)(S) = (K)(W)(D_w)\frac{dh}{dl}$$

Where Q is flow rate in m^3/d, K is hydraulic conductivity in $m^3/m^2.d$, A_c is flow cross sectional area in m^2, i.e. (W) x (D_w), W is width of VSB in m, D_w is water depth in m, L is length of VSB in m, dh is head loss in m and S is hydraulic gradient. This form assumes laminar flow through the media; K varies with time and location within the media and has major impact on the head loss. Dimensions of the VSB are derived from Darcy's law. European guidelines also recommend hydraulic slope to be provided according to Darcy's law. Further, the guidelines suggest use of hydraulic conductivity no greater than that of the original media being used. Hydraulic conductivities of different materials are shown are shown in Table 7.5.

Table 7.5: Hydraulic conductivity values for different type of media

Media type	Hydraulic conductivity in $m^3/m^2.d$
Medium sand[1]	420
Coarse sand[1]	480
Gravelly bed[1]	500
5-10 mm gravel[2]	34000
20-30 mm gravel[2]	100000

(Source: [1] Wood, 1995 and [2] EPA, 1999)

Brodie et al., 1988 have observed no significant differences in the treatment efficiencies, when different substrates were used as bed materials. Hence, locally available bed materials may be preferred for articulating the wetland systems. In order to avoid clogging of the bed and the associated problems, coarse materials should be preferred. In this design it is proposed to use 20-30 mm gravel for which hydraulic conductivity can be taken as 100000 m/d. It is assumed that hydraulic conductivity of initial final treatment zone stabilizes at 1% and 10% respectively of this value (EPA, 1999).

Surface area of a VSB system can be determined by number of ways:

(i) "Keckuth equation" (Cooper and Green, 1995):

$$A_h = \frac{Q_d (\ln Ci - \ln C_e)}{K_{BOD}}$$

Where A_h is bed area required per population equivalent (pe) in m^2, Q_d is daily average flow rate in m^3/d, C_i is daily average influent BOD in mg/l, C_e is daily average BOD desired in the effluent in mg/l and K_{BOD} is rate removal constant in m/d. Green and Upton (1995) recommend A_h value of 5 m^2 / head.d.

(ii) EPA (1993) equation for calculating the bed surface area:

$$A_s = LW = \frac{Q \ln \frac{Ci}{Ce}}{K_t dn}$$

Where A_s is bed surface area in m^2, n is effective porosity of media, L is length of bed in m, W is width of bed in m, Q is average flow in m^3/d, C_i is influent BOD in mg/l, C_e is effluent BOD in mg/l, d is depth in m and K_t is a rate constant in d^{-1}.

(iii) Loading criteria for calculating bed surface area (EPA, 1999)

In the present design BOD areal loading criteria (suggested by EPA, 1999) has been used

First order kinetics such as the one given below for BOD removal can be used to calculate hydraulic retention time required for a VSB:

$$\frac{Ci}{C_e} = e^{-k_t t} \qquad \text{(Crites, 1994)}$$

Where C_e and C_i is effluent and influent BOD in mg/l, t is hydraulic retention time in days and K_t is first order reaction rate constant in d^{-1}. Typical detention time for VSB systems ranges from 2 to 7 days. For avoiding entry zone clogging and subsequent surface flow, BOD and suspended solids loading should be kept low. Overall BOD loading for VSB wetland systems should not exceed 75 kg/ ha.d (Crites, 1994). For VSB wetland systems, bed width is determined by hydraulic flow rate and length is determined by the hydraulic retention time required for pollutant removal. Therefore, VSB wetland system may have an aspect ratio less than or greater than 1:1 depending upon the treatment goal. Media depth may range from 0.3 to 0.75 m, and the media particle size for most gravel VSB systems is in the range of 5 to 250 mm with 13 to 76 mm as typical range. Practically neither of these models/equations is actually used in the design of VSB wetland systems. Instead, the systems are designed using the design criteria.

Design criteria suggested by Wood (1995) and EPA (1999) are summarized in table 7.6 and 7.7 respectively.

Table 7.6: Process design criteria for VSB constructed wetland systems

Factor	VSB
Detention time, d	2-7
Max BOD loading rate, kg/ha.d	75
Water or substrate depth, m	0.1-1.0
Hydraulic loading rate, mm/d	2-30
Areal requirement, ha/ m^3.d	0.001-0.007
Aspect ratio	0.25:1 to 5:1
Harvest frequency, yr	3-5

(Source: Wood, 1995)

Table 7.7 : Summary of VSB Design criteria

Parameter	**Criteria**
Surface area based on desired effluent quality and areal loading rates as follows	
BOD	6 g/m^2 –d (53.5 lb/ac-d) to attain 30 mg/l effluent
BOD	1.6 g/ m^2-d (14.3 lb/ac-d) to attain 20 mg/L effluent
TSS	20 g/ m^2-d (178 lb/ac-d) to attain 30 mg/L effluent
TKN	Use another treatment process in conjunction with VSB
TP	VSBs not recommended for phosphorus removal
Depth Media (typical)	0.5 – 0.6 m (20-24 in.)
Depth Water (typical)	0.4 – 0.5 m (16 – 20 in)
Length	As calculated; minimum of 15 m (49 ft)
Width	As calculated; maximum of 61 m (200 ft)
Bottom slope	0.5 – 1%
Top slope	Level or nearly level
Hydraulic Conductivity	
First 30 % of length	1% of clean K
Last 70% of length	10% of clean K
Miscellaneous	Use at least two VSBs in parallel Use adjustable inlet device with capability to balance flows Use adjustable outlet control device.
Media	
Inlet zone (first 2 m)	40-80 mm
Treatment zone	20-30 mm
Outlet zone (last 1 m)	40-80 mm

(Source: EPA, 1999)

(c) Design Calculations

A. Field design data

Design discharge = 837.73m^3, BOD in clarified sewage = 80 mg/l

TSS in clarified sewage = 202.95 mg/l, Desired BOD in effluent = 30 mg/l

B. Data derived from design criteria

Areal loading rate for BOD = 75 kg/ha.d = 7.5 g/m^2.d

Areal loading rate for TSS = 200 kg/ha.d = 20 g/m^2.d

Bottom slope = .005, Bed media = 20-30 mm rounded gravel

Hydraulic conductivity for assumed bed media K = 1,00,000 m/d

Hydraulic conductivity for initial treatment zone = 1% of K= 1000 m/d

Hydraulic conductivity for final treatment zone = 10% of K = 10000 m/d

Water depth at inlet = 0.4m, Media depth = 0.60 m

Maximum allowable head loss through initial treatment zone

=10% of media depth = 0.06

C. Design calculations

(i) Surface area, bed dimensions and number of cells

Surface area based on surface loading rate= $\frac{837.73x80}{7.5}$ = 8935.78 say 8936 m^2

Surface area based on TSS loading rate = $\frac{837.73x202.95}{20}$ = 8500.86 m^2

Hence adopt a surface area of 8936 m^2

Surface area for initial treatment zone A_{si} = 30% of total area = 2680.8 say 2681 m^2

Surface area for final treatment zone A_{sf} = 70% of total area = 6255m^2

Width required for head loss of 0.06 m and using Darcy's law

$$Q= K_i \text{ x } W \text{ x } D_{wi} \text{ x } dh_i/L_i$$

$$L_i = \text{length of initial treatment zone} = A_{si}/ W$$

Solving for $W^2 = \frac{QxA_{si}}{K_i x dh_i x D_w} = \frac{837.73x2681}{1000x0.06x0.4} = 93581$

W = 305.91 say 306 m

It is proposed to provide 18 cells of 17 m wide <61 m hence OK

L_i = 2681/306 = 8.8 m

length of final treatment zone L_f = 6255/306 = 20.5 m

Assume length of bed = 30 m

Head loss in initial zone $dh_i = \frac{837.73x8.8}{1000x306x0.4} = 0.06$ (as assumed)

Head loss in final zone $dh_f = \frac{837.73x8.8}{1000x306x0.4} = 0.014$

(ii) Bottom elevations

Assuming bottom elevation at outlet as 0 (reference), elevation of bottom at beginning of final treatment zone E_{bf} = (slope) x Length = 0.005 x 20.5 = 0.10 m

Elevation of bottom at inlet E_{bi} = (slope) x Length = 0.005 x (8.8+20.5) = 0.15 m

(iii) Water surface elevations

Elevation of water surface at the beginning of final treatment zone

$E_{wf} = E_{bf} + D_w = 0.10 + 0.4 = 0.50$ m

Elevation of water surface at outlet

$E_{wo} = E_{wf} - dh_f = 0.50 - 0.014 = 0.486$ m

Elevation of water surface at inlet

$E_{wi} = E_{wf} + dh_i = 0.50 + 0.06 = 0.56$ m

(iv) Water depths

Depth of water at inlet $D_{wi} = E_{wi} - E_{bi} = 0.56 - 0.15 = 0.41$ m

(approximately equal to assumed depth hence OK)

Depth of water at beginning of final treatment zone $D_{wf} = E_{wf} - E_{bf} = 0.5 - 0.1 = 0.4$ m

Depth of water at outlet $D_{wo} = E_{wo} - E_{bo} = 0.486 - 0 = 0.486$ m

(v) Determination of media depth

It is assumed to provide a level surface at top. To have the level surface the media elevation shall be greater than the highest water elevation i.e. at inlet (0.56 m).

Check for the depth of media over the water surface assuming media depth as 0.60

Depth of media at inlet = 0.60 – 0.15 = 0.45 m

Depth of media at beginning of the final treatment zone = 0.60 – 0.1 = 0.5 m

Depth of media at outlet = 0.60 – 0 = 0.6 m

Depth of water from top surface at inlet = 0.6 – 0.56 = 0.04 m

Depth of water from top surface at beginning of the final treatment zone

= 0.6 -0.5 = 0.1

Depth of water from top surface at outlet = 0.6 – 0.486 = 0.114 m

Since desirable depth of media over water surface should be around 0.1 m less depth of media in inlet zone is compensated by providing an additional layer of media at first few meters of the bed.

Design check with reference to criteria given in Table 7.6

Detention period = $\frac{8936x0.4}{837.73}$ = 4.27 days (should be between 2-7 days hence OK)

Surface area provided / m^3 .d = $\frac{8936}{837.73}$ = 10.67 m^2/m^3.d = 0.001 ha/m^3.d

(should be between 0.001 -0.007 hence OK)

Aspect ratio (length to width ratio) = 30 : 18 = 1.67 : 1

(should be between 0.25 : 1 to 5 : 1hence OK)

7.3 Disposal System

(a) Design Considerations

(i) Since the irrigational water requirement depends on climatic conditions and crop pattern a suitable arrangement (during learn period) to discharge the water in the road side drain has been considered. This system takes care of the rainfall as well as catchments runoff water and helps in maintaining the fixed maximum water level under critical conditions (i.e. lean period, runoff, rainfall etc).

(ii) The fish pond has been designed to store 10 days wastewater with an average depth of 1.5 m.

(iii) Two pumps (one in operation and one as stand by) of suitable capacity has been provided to pump clarified water. Suitable provisioning for disinfection should be provided.

(b) Design Calculations

The capacity of fish pond = 837.73 x 10 = 8377.3 m^3

Assuming 1.5 m depth, surface area required = 5584.86 m^2

Assuming circular pond diameter of pond = 84.32 m

Provide a pond of 85 m diameter with overflow weir discharging the water in to the open drain.

CHAPTER 8

CONCLUSION

In the present study a scheme for the management of sewage generated in a medium size cantonment has been developed. Further, important units of the scheme have been designed for a specific case (cantonment).

The scheme developed is different from the conventional scheme used for sewage management. Salient features of this scheme are:

(i) Providing intercepting tanks for clarifying the sewage, prior to its entry into the sewers– This feature is supposed to minimize sewer clogging problems a common feature in the cantonments sewerage system.

(ii) Using a vegetated submerged bed constructed wetland system for secondary treatment of the sewage rather than using any conventional treatment technology– Constructed wetland system is considered to be maintenance free and involve no operational difficulties further, it will not demand already scarce energy resources and hence it is assumed to superior to other used technologies.

(iii) The scheme also includes ASP (which is a commonly used conventional technology) as an alternative option to constructed wetland system– The idea behind including ASP is to provide opportunity to the decision maker to compare it with constructed wetland system and assess the latter's superiority over the former.

The small bore sewerage system which is claimed by the student as superior to the conventional sewerage systems can have problems may be much more serious if the intercepting tanks are not properly managed. Intercepting tanks are supposed to retain the solids of the sewage received and allowed only the clarified sewage into the sewer. The sewers are in fact designed for carrying clarified sewage rather than raw sewage. If intercepting tanks are not cleaned at the intervals for which they are designed there is a danger of overflowing of the solids along with the sewage in to the sewer and thus clogging it.

Though clarified sewage is received at the sewage treatment plants, since the efficiencies of suspended solid removal in the intercepting tank can not be that high, around 40 to 100 mg/l suspended solid can still be expected in the sewage received at the sewage treatment plant. All these solids are expected to get accumulated in the initial section of the constructed wetland system. This accumulation over a period can lead to clogging and flooding of the wetland bed. A flooded wetland bed is undesirable because of intense flies and mosquitoes problems, for avoiding such problems the initial sections of the bed may be left unvegetated and relatively larger size gravel stones or media may be used in the articulation of bed and if need arises the initial section may be overhauled as and when required.

Maintenance required for small bore sewerage system, though very limited may require a deferent set of appliances and equipments in fact these may require purging at high pressure with water.

As of now constructed wetland system technology is almost not used in India and the design, operation and control parameters for constructed wetland system are not developed sufficiently. The design provided in the study has been based on criteria and assumptions proposed for temperate claimate like USA and Western Europe. With the experience gained and through organized and structured monitoring of the system over the time information may accumulate and help in having design, operation and control parameters for Indian conditions.

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