EXPLOITATION OF SOFTWARE
IN
DESIGN OF UNDERGROUND SEWERAGE
SCHEME WITH A CASE STUDY

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BY

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CERTIFICATE

Certified that thesis entitled “EXPLOITATION OF SOFTWARE IN DESIGN OF UNDERGROUND SEWERAGE SCHEME WITH A CASE STUDY” which is being submitted by Mr. Deepak Lakhanpal in partial ftilfillment for the award of the degree of MASTER OF ENGINEERING IN CIVIL ENGINEERING (ENVIRONMENTAL ENGINEERING) OF THE UNIVERSITY OF DELHI, is he record of student’s own work carried out by him under my supervision and guidance from March 2001 to January 2005. The matter embodied in this thesis has not been submitted for the award of any other degree.

(‘Prof H.S. Bhatia)

Guide

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SYNOPSIS

The project entitled “**EXPLOITATION OF SOFTWARE IN DESIGN OF UNDERGROUND SEWERAGE SCHEME WITH A CASE STUDY**” deals with the approach required for planning and design of underground sewerage scheme. The report discusses topographical survey requirements, planning and designing aspects of underground sewerage scheme, design approach of computation of economical diametre of Pumping Mains, Wet Wells and Pumping Machinaries, types of manholes provided in different conditions, design of sewage treatment plant and project implementation.

A case study of underground sewerage scheme of Silvassa has been done with typical sample computations of design of sewer network , economical diametre of pumping main, wet well, pumping machinary and sewage treatment plant.

The design of sewer network has been done by “ SEWER” software which is based on Manning’s formulae. For the purpose of network analysis, entire sewer line was transferred to link-node network. The “SEWER” software is found to be very helpful in providing quick computations of total sewage flow at each node by considering the peak factor and ground water infiltration, design diameter of sewer line, velocity generated in each of the sewer link, upstream and downstream crown levels, invert levels and excavation depths of the sewer links, d/D and pipe slope of each of the sewer link.

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CHAPTER - I

INTRODUCTION

1.1 NEED FOR SEWERAGE SCHEMES

Sewage is a water borne waste containing suspended and dissolved solids of organic & inorganic nature. It contains great variety of biological life e.g. viruses, bacteria, worms, insects, algae & fungi etc. many of whom are pathogenic to human and animal life. Therefore, sewerage schemes are required to effectively treat the sewage to required degree and safely dispose the effluents as per prescribed norms/ standards. BOD and suspended solids of effluent after treatment should not be more than 30 mg/l & 50 mg/l respectively.

1.2 STATUS OF WATER SUPPLY AND SEWERAGE DISPOSAL

Sewerage schemes cannot function if the rate of water supply is not adequate. Therefore to design a sewerage scheme, existing status of water supply and future water requirement for the design period of sewerage scheme has to be studied in detail. Status of sewerage disposal viz through septic tank/soak pits/underground sewerage scheme is necessary for proper comprehension of problem.

1.3 TOPOGRAPHICAL SURVEY REQUIREMENT

Detailed topographical survey of the project area is the basic requirement for formulation of any sewerage scheme. It is therefore, essential to carry out detailed survey of the project area to map contours, the main roads, laterals, branch roads, location of built up, residential areas, commercial areas, industrial areas, playground, open spaces, natural drainage courses, man made drainage channels, cross drainage structures etc. Contour plans, and spot levels along roads are some of the necessary requirement for planning and designs of sewers.

1.3.1 Reconnaissance Survey

Reconnaissance Survey needs to be carried out in order to have full comprehension of the project area, sewage treatment plant site and to select outfall point for disposal of sewage.

1.3.2 Height Control

G.T.S. benchmark established in or near the study area should be identified to carry out the survey work. A series of temporary bench marks should be established at an interval of about 0.5 – 1 Km for detailed survey of project area.

1.3.3 Main Lines

Main levelling line should be run along all the main roads carrying levels from the reference bench mark to establish bench marks at 0.5 km interval.

1.3.4 Subsidiary Lines

Subsidiary levelling lines should be run along branch roads to establish bench marks at all control points in order to establish height control at closer intervals.

1.3.5 Tie Lines

A number of tie lines commencing from subsidiary and main levelling lines should be traversed to provide levelling heights at very close interval for carrying out subsequent contouring at 0.5 meter interval.

1.3.6 Compilation of Base Maps

Base maps drawn on 1:2500 /1:5000 scale depending on the extent of study area should show various features as given below:

(i) All roads, streets, lanes, Locality and sub-locality with their names

(ii) All culverts and bridges with their dimensions and height

(iii) Important buildings, govt. offices, colleges, worship places, factories, Ponds, tanks, temples, streams, rivers with the names.

(iv) Open areas, plantation/cultivation limits, play grounds, overhead water tanks etc.

(v) Bench mark location with their RLs

1.3.7 Contouring

Levelling should be carried out during main, branch and tie line survey while their RLs correctly verified and plotted on base maps. Contour interval of about 0.50 m should be taken for plotting of contours on base maps.

1.4 DATA REQUIREMENT

Following data/information is required for planning and design of sewerage scheme.

a) Demographic profiles

b) Development Plan showing the demarcation of industrial, commercial, residential areas, roads, highway, railway alignment, forest areas, community facilities

c) Hydrological data viz position of ground water, soil strata

d) Hydrological studies viz river flows, fluctuations

e) Rate of water supply (present and future)

f) Meteorological information viz prevalent wind directions

g) Socio-economic aspects, affordability of water and sewerage services

CHAPTER – II

DESIGN SCENARIO

2.1 DESIGN PERIOD

Generally sewerage schemes are designed for a period of 30 years excluding construction period. Sewer pipes and sewage treatment plants are designed for design period of 30 years. For economical design, sewage treatment plants are initially designed for 5-10 years with provision for future expansion. Design period for pumping machineries is generally 15 years.

2.2 POPULATION PROJECTION

As mentioned above, different components of the proposed system should be designed for 15 year/30 year design periods. Accordingly population of the town should be projected for base year, intermediate stages and ultimate stage. The projected population estimates should be assessed by different methods viz (a) Arithmetical increase method (b) Incremental increase method (c) Geometrical Increase method and (d) Semilog method and suitable method of population forecasting should be adopted after carefully studying the past growth pattern of the city.

Policy of the concerned state government viz special incentives offered to establish industrial units in urban area should be taken into account while forecasting population as industries create direct as well as indirect employment and, therefore, attract people from other regions specially the skilled labour. The population of the region is thus bound to increase substantially due to migration besides the natural increase.

2.3 PER CAPITA SEWAGE FLOW RATE

Sewage contribution from communities largely depends on the level of water supply provided to the communities. The quantity of water flowing into a sewer system is normally less than the quantity of water supplied to the community. Generally, water supplied to the community is assumed to be discharged in sewers @ of 80% as per CPHEEO manual. Commercial, industrial and institutional establishments often have their own water supply arrangements and discharge the wastewater into municipal sewers.

2.4 GROUND WATER INFILTRATION

As the sewers are laid underground and the joints made are not hundred percent watertight, infiltration of ground water through the joints can not be ruled out. Besides there is likelihood of entry of surface water through manholes or abandoned house connections and, therefore, allowances for ground water infiltration into the sewer should be taken into account as prescribed by CPHEEO manual.

2.5 PEAKING FACTORS

Fluctuation in the rate of sewage flow from any sewerage zone varies with season, month, day and hour. Maximum or peak flow and minimum flow are the controlling factors in design of sewers. The sewers should be so designed as to cope up with the peak flow and as well as to develop self-cleansing velocity for minimum flow. To determine the relationship between average and maximum flow rate, a few empirical formulae exist but the result can not be relied upon. Gauging of sewage flow rate of a similar town can only give realistic picture, which can be considered for adoption. But in absence of any such data, recommendations of CPHEEO manual relating to peaking factor i.e. ratio of maximum flow to average flow, based on contributory population, should be adopted as given below.

Contributory Population		Peaking Factor
Upto 20,000		3.0
20,000 to 50,000	-	2.5
50,000 to 7,50,000	-	2.25

2.6 SELF CLEANSING VELOCITY

Self cleansing velocity depends on size and specific gravity of the particle to be transported. CPHEEO manual recommends a velocity of 0.6 m/s for liquids containing particles of size 0.09 mm and specific gravity of 2.65. However, it is difficult to attain this velocity at initial stage of scheme, in starting reaches if the population of the study area in the initial stage is low.

If the industrial wastes are not included in the design of sewage, suspended solids are likely to be of smaller size and low specific gravity. Therefore, a minimum velocity of 0.35 m/sec can be kept as self cleansing velocity, whereas in trunk sewer the velocity developed should be 0.60 m/sec for present flow and 0.70 to 1.12 m/sec at designed peak flow. Links, which develop a velocity less than 0.35 m/s, facilities for flushing with help of mobile tankers, should be recommended.

CHAPTER – III

PLANNING AND DESIGN OF SEWER NETWORK

3.1 GENERAL

Planning of sewer network is done in such a way that population in every part of the project area is provided with sewer links to collect sewage from that area and transport it to Sewage Treatment Plant by way of gravity/pumping.

Keeping in view the topography of the project area, it should be divided into a suitable number of zones so that sewer network can be planned in such a way that sewage generated in that zone can be collected at its outfall point by way of gravity. Pumping of sewage should be resorted to keeping in view low lying areas, limitation of depth of excavation and location of sewage treatment plant.

3.2 SEWERAGE ZONES

According to topography of town, entire project area should be divided into suitable zones by taking into account future development of infrastructural facilities in the region viz residential/commercial/institutional complexes, roads etc. While implementation of the scheme, developed area should be taken in first phase whereas undeveloped and low lying areas should be taken-up in Phase II. The implementation of Phase II shall be taken-up as and when development in these area takes place.

3.3 LAYOUT OF SEWER NETWORK

Circular RCC sewers shall be laid underground in proper slope to collect the sewage from contributory population of sub zones and carry it under gravity to point of outfall. Direction of flow of sewage should be shown by an arrow marked on the sewer line. Sewers shall be laid on opposite side of the road carrying drinking water mains to avoid any contamination. The trunk sewers should be proposed along main roads whereas branch sewers and laterals should be laid along the branch and lateral roads respectively.

Various types of manholes should be provided to facilitate cleaning and inspection of sewers, depending upon depth of sewers. They are provided at each junction of sewers, change of grade, change in direction, at place of drops and at 30 m interval on straight runs. Rectangular manholes should be proposed upto depth of 0.9 m below ground level and beyond this depth circular brick manholes should be proposed. Drop manholes should be proposed to minimise turbulence where fall is more than 0.6 m.

For the purpose of network analysis by sewer software, entire sewer network can be transferred into a link-node network.

3.4 DESIGN OF SEWER NETWORK

Design of sewers can be done using 'SEWER' software. Manning's formula has been used for design of sewers. Various design parameters adopted for designing sewer network of various zones are given below:

Minimum velocity in sewers	:	0.6 m/s
Maximum velocity in sewers	:	1.1 m/s
Maximum d/D for sewer	:	0.80

Minimum allowable cover for sewers	:	1 m
Maximum allowable cover for sewers	:	6 m
Ground water infiltration rate	:	13.46%
Manning's coefficient (n)	:	0.013
Peak factors	:	In accordance with the serving population of the zone.

Drops should be provided wherever necessary. Flushing through mobile tankers shall be resorted to in the sewer links in which velocity is less than 0.60 m/s. In reaches where excavation depth is between 0.6 m to 1 m concrete encasement shall be provided over the sewer links to protect these from damage due to vehicular traffic. For sewer links which shall be laid below ground water table, concrete cradle bedding should be provided.

CHAPTER - IV

DESIGN OF PUMPING MAINS, WET WELLS AND PUMPING MACHINARIES

4.1 PUMPING MAINS

Depending upon the topography of the study area and in order to avoid laying of sewers in deep excavation below ground level, pumping of sewage can be resorted to at suitable locations. Economic sizes of pumping mains can be computed in accordance with guidelines prescribed in the CPHEEO manual taking into account capitalisation of the overall investment required for cost of pipe line, energy requirement for pumping the raw sewage against frictional losses and static head, pumping machineries, choosing different sizes of pipes and comparing the relative cost for adopting the economical size.

Sluice valves should be provided at suitable locations along the alignment to facilitate smooth operation and maintenance. In order to avoid any likely chance of contamination of drinking water the sewage pumping mains are proposed to be aligned across the opposite flank of the road housing water pipe mains.

4.2 WET WELLS

The function of the sump or the wet well is to balance the difference between the rates of inflow of sewage to the pumps and the rate of its discharge by pumping. Since it is impossible to adjust the rate of pumping exactly equal to the unknown rate of inflow such a provision is very essential. The storage capacity of the wells should be designed considering the time for which the sewage is to be retained and frequency of pumping operation. Detention time is so kept that deposition of solids may be avoided and septic conditions may not develop. Maximum detention time is not to exceed thirty minutes of average flow.

4.3 PUMPING MACHINERIES

Selection of pumping machineries can be done considering the availability of land, site conditions, quantum and characteristics of sewage. Pumping stations with submersible pumps are economical as the dry pit for installation of pump and superstructure for protection of motor are not required. Besides, the problem of land acquisition are not encountered on account of underground installation of entire pumping machineries.

Rating of pumps installed in pumping stations can be determined by computing peak sewage flow coming into the pumping station, static head, terminal head, frictional losses in the pumping mains and losses in the pipe fittings.

CHAPTER - V

MANHOLES

5.1 GENERAL

Manholes are constructed on the alignment of sewers to facilitate testing, inspection and cleaning of sewer pipes. Manholes are provided at each junction, change of alignment and grade.

5.2 RECTANGULAR MANHOLES

Rectangular manholes of size 900 x 1200 mm can be provided on sewers laid 0.9m below G.L. as per recommendation of CPHEEO manual.

5.3 CIRCULAR MANHOLES

Circular manholes having internal diameter of 900 mm are provided for depths above 0.9 m and upto 1.65 m, 1200 mm for depths above 1.65 m upto 2.30 m and 1500 mm for depths above 2.30 m upto 9.0 m.

All the manholes mentioned above should be constructed in brick masonry, slanting in top portion, in order to have an opening equal to external diameter of the manhole cover. The R.C.C. manhole frame and cover can be provided as it is economical in comparison to CI frames. C.I. steps should be provided in each manhole for easy descent of maintenance personnel.

5.4 R.C.C. MANHOLES

R.C.C. Circular manholes of internal diameter of 1500mm should be provided for depth above 4.3 m upto 7.3 m and where high subsoil water conditions are encountered.

5.5 DROP MANHOLES

Vertical drop pipes are provided wherever level difference between peak flow level of main line and invert level of the branch line exceeds 600 mm. The vertical drop pipe is supported on W.I. brackets inside the shaft.

CHAPTER – VI

DESIGN OF SEWAGE TREATMENT PLANT

6.1 GENERAL

Various aspects must be considered while planning waste treatment facilities. These considerations may be classified as under

- Environmental
- Engineering and site
- Process, performance, energy
- Cost

Planning for waste water treatment facilities generally begins from the final disposal site going backwards in order to obtain an integrated and optimum design which suits the topography and the available hydraulic head.

6.2 Environmental impact considerations

For any major investment infrastructure, having clear environmental and socio-economic impacts,it is desirable to carry out an Environmental Impact Assessment (EIA) of the proposed treatment plant and its disposal works preferably before a site is finalised. Following points should be considered in EIA.

- Downstream hydrology and aquatic eco system
- Ground water pollution potential
- Estuarine and coastal waters; mangroves and wet lands
- Odours, aerosol sprays, mosquito nuisance

- Effects on nearby land use and land values
- Rehabilitation of affected people
- Public health aspects which includes discharge standards, toxic in food chains, bathing water quality, chemical pollution of water resources
- Health of plant operators/ farm workers
- Plant operation and maintenance aspects
- Green belt and landscaping

6.3 Engineering and Site Considerations

Following engineering considerations must be considered in site selection and project design

- Topography of general area to be served, its slope and terrain. Tentative site suitable for treatment plant location, intermediate and final pumping station. Contour plan of the tentative sites for detailed layouts.
- The ultimate design of the sewage treatment plant should be done for a period of 30 years. But in the first stage plant should be designed for a period of 10 to 15 years with its related machinery.
- Available hydraulic head in the system up to high flood level in case of disposal to a river; high tide level in case of estuarine or coastal disposal; level of the irrigation area to be commanded in case of land disposal
- Ground water depth and its seasonal fluctuations affecting construction of the proposed site, structural design, sewer infiltration and land disposal facilities of treated effluent
- Soil bearing capacity and type of strata expected to be met in construction at proposed site.

6.4 Process, Performance, Energy, Operation and Maintenance

Process considerations involve factors which affect the choice of treatment method, its design criteria and related requirements as given below

Wastewater flow and characteristics : This constitutes the primary data required for process design

Degree of treatment required : The selection of treatment process depends on degree of treatment required in terms of removal of various parameters such as BOD/ COD, coliforms, helminths, nutrients etc. Land disposal, generally has to met less stringent discharge standards than disposal to surface water. Land disposal has the advantage of giving nutrient removal and is, thus preferred wherever it is feasible.

Performance fluctuations and dependability : Waste water fluctuates during day and night. Different treatment processes have different performance fluctuation characteristics. The dependability of performance of a process inspite of fluctuations in influent quality and quantity is very useful feature while selecting the type of treatment plant.

Other process requirements : Various other factors affecting the choice of a process are given as under

Land : Depending upon the availability of land in the selected site, choice of treatment system is made. Activated sludge process which requires little land is useful in the big cities as the price of the land is one of the governing factor there.

Power : Operation and maintenance cost of the treatment system depends on the energy requirements of the system. Ability of a process to withstand the power failures is great asset.

Skilled staff : Availability of skilled staff in the region is the important criteria while selecting the type of treatment method to be adopted

Nature of maintenance problems

Extent of sludge production and its disposal requirements

Loss of head through plant in relation to available head

Ease of stagewise extension of plant with time

sludge process, trickling filters etc. are not conducive to the requirement of the undeveloped town, in view of high cost and sophisticated operability of the treatment plant. Facultative aerated lagoons should be used in such conditions due to their simplicity in operation, and minimum need of machineries.

CHAPTER –VII

COST ESTIMATES

The cost estimate of different components of the sewerage scheme should be based on the prescribed SOR (Schedule of Rates) of the concerned state.

The capital cost of various items of work includes Sewer Collecting System, Pumping Stations, Sewage Treatment Plant, Power Supply, Staff Quarters, House Connections, Land Acquisition, Lab. Building & Testing Equipment, Vehicles etc.

Annual operation & maintenance expenditure on the project should also be calculated in accordance with the standard yard stick of maintenance staff and management personnel and necessary provision for spares of pumping machinery, electrical equipments tools etc. It also includes maintenance cost of civil structures, mechanical parts of water treatment plant, cost of annual energy charges for running of pumps, aerators etc.

CHAPTER – VIII

PROJECT IMPLEMENTATION

8.1 IMPLEMENTATION

Sewerage projects require sophisticated technical skill and experience for proper execution in accordance with designs and drawings prepared for the project. Therefore the primary responsibility devolves on the technical personnel who are entrusted with execution of different components of the project to follow standard technological practices while implementing the project. Procedures and practices to be followed in implementation of layout of collecting system, wet wells, pumping stations, Sewage treatment plant and other sewer appurtenances are outlined in this chapter.

8.2 LAYING OF SEWERS

As the sewers are laid underground in line and grade, excavation of earth to the required depth and to keep the sewer trenches within limits of technical and economical requirements, are important factors to be taken care of while laying sewer pipes. Sewer trenches should be excavated as narrow as possible for laying and jointing the sewers, inspecting the joints, refilling of the trenches and compacting the refill to the required degree. In deep trenches upper section should be cut wider than the lower sections or the sides of the trench should be sloped to minimize the possibility of caving. Width of trench as per Indian standard specifications should be equivalent to external dia of pipe in cm + minimum 40 cm working space + additional space of 10 cm all round of additional depth per metre. The bedding materials should be laid and compacted to proper grade and shaped to fit the bottom of the pipe. It is not desirable to keep open trenches for more than 6 to 7 days in advance of pipe laying because most soils cave in and traffic hazards due to open trenches are prolonged. safety precautions must be made to safeguard against accidents. Such precautions include the erection of signs, barricades, maintenance of lights for illumination and danger signals during night time as well as even providing night watch men to exclude unauthorized persons particularly children from trespassing on the work. Before commencing excavation stakes should be driven along the centre lines of proposed alignment of sewers at regular intervals. Manholes and other similar structures where wider excavations are required should be located by two pairs of reference stakes so that string joining the reference points on each pair of stakes should intersect approximately at right angles over centre of the structure and, thereafter, the shape of the excavation should be laid on the ground.

8.3 SHORING OR TIMBERING

To prevent caving of side walls of the trenches shoring or timbering is required to be done. Wooden boards should be placed vertically against the sides of the trenches suitably cross braced and wedged in place to hold the vertical boards in position. The boards and cross braces should be placed in position after the trenches have been excavated.

8.4 TRANSFERRING LINES AND ELEVATION TO PIPE IN TRENCH

Grade of a sewer is the most important element that causes gravity flow in the sewer at designed velocity and discharge and hence satisfactory functioning of the entire system depends on it. Therefore, extreme precaution needs be taken while transferring the invert levels of pipes laid in trenches. Horizontal boards spanning the trenches are fixed by nails driven in stakes on both sides of the trenches at intervals of 7.0 metre. The center line of the sewer is marked on the cross piece and the top of the cross piece is fixed at the required grade by means of levelling instruments. After fixing the sight rails at required elevations the levels are transferred by means of boning rods with a right angled projection at the lower end by marking with chalk or a notch at such a distance from the end that when mark is held as the centre of the cross piece the lower portion of the rod that projects into the pipe will rest on the invert level as required.

8.5 BACK FILLING

It is necessary to exercise care in backfilling the trenches to prevent displacement of newly laid pipe and to avoid subsequent settling of the surface. The backfilling should be done as soon as the cement or other construction and joint materials have properly set. Where timbering has to be withdrawn it should be done immediately before the commencement of backfilling. The backfilling should be done evenly on both sides of the pipe and tampering should be done continuously but no tampering should be done within 150 mm of the top of the sewer. If in deep excavations high subsoil water condition is encountered, cement concrete (1:2:4) should be provided in bedding instead of murrum.

8.6 JOINTING OF R.C.C. SPUN PIPES

R.C.C. circular hume pipes of NP_2 Class pipes with collors have been proposed upto 500 mm dia and NP_3 above it. Collar joints in cement mortar (1:3) should be done. Care should be taken so that concentricity and levels are not disturbed during this operation.

8.7 HYDRAULIC TESTING OF PIPE SEWERS

Hydraulic testing to ensure the water tightness of the sewer joints should be done from manhole to manhole as per procedures outlined in CPHEEO manual. The leakage to maintain the test pressure during the period of 10 minutes should not exceed 0.2 liters/mm dia of pipe/km length/day.

8.8 TEST FOR STRAIGHTNESS OF SEWERS

To check the straightness of the sewer line, there is one simple test that can be easily exercised. A light source is put inside the preceding manhole near the mouth of the pipe and its image on a white cardboard screen inside the following manhole is observed. If the image is perfect circle then it shows that there is no kink in the pipeline between the manholes and the sewer is laid straight. This simple test should be carried out from manhole to manhole after laying the sewers to ensure straightness of sewers.

8.9 SUB SURFACE CEMENT CONCRETE STRUCTURES

Underground circular R.C.C. wells have been provided with submersible sewage pumps for sewage pumping at various locations within the Project Area. The wet wells extending beyond 5.0 m below ground level are likely to encounter high sub-surface water conditions. Chemical analysis of ground water sample have shown high contents of Sulphates and Chlorides which are quite corrosive for cement concrete structures. Therefore, suitable Portland Puzzolana cement should be used in the construction of underground R.C.C. structures as per advice of the manufacturing companies. Treated reinforcement steel may also be used to safeguard against corrosion. Water proofing chemicals should be used in proportion as advised by manufacturers in constructions of sump wells to ensure water tightness. The number of construction joints should be limited as far as possible. Cover to steel reinforcement at water faces should not be less than 37 mm considering the corrosive water existence. All the piping inside and out side pumping stations up to the point of pumping mains, sluice valves should be flanged pipe jointed with rubber packing and nuts & bolts.

CHAPTER-IX

A CASE STUDY OF SILVASSA SEWERAGE SCHEME

9.1 SALIENT FEATURES OF THE PROJECT AREA

Silvassa town is the capital of Union Territory of Dadra & Nagar Haveli. The adjoining village of Amli forms part of Silvassa. Combined area of Silvassa and Amli is about 17 Sq.km. The town is centrally located in the Union Territory at 73^o 30′ E longitude and 20^o 16′ 30″ N latitude. The nearest railway station is Vapi on Mumbai - Ahmedabad broad gauge section of Western Railway which is about 20 km from Silvassa. Silvassa is well connected by good roads with the other villages of Union Territory and towns of neighbouring state of Gujarat.

The topography of the town is marked with many undulations of levels, the variation in maximum & minimum levels of town is of the order of about 15 m. The rainy season occurs in months of June to September. Average annual rainfall is about 2200 mm. The temperature normally varies between 15^o to 35^oC.

As per 1991 census population of Silvassa is 24608 persons. Keeping in view considerable industrial development taking place in Silvassa, Town & Country Planning Department has projected a figure of 124668 to be the population of the area in 2021 (Regional Plan – 2020).

As per existing landuse pattern, land of about 136.7 ha. is being used for residential/commercial proposes. An additional area of about 323 ha. is proposed to be covered under residential/commercial uses in future thus totaling to about 460 ha. (27% of total area).

9.2 TOPOGRAPHICAL SURVEY

For the purpose of preparing sewerage scheme in Silvassa, detailed topographical surveys were carried out. G.T.S. benchmark near foot of pillar of existing bridge on Piparia river with R.L. of 31.50 m was used as reference point for levelling in the project area. 65 nos. bench marks were established within the project area referred to permanent objects at distance of 0.5 km interval almost at all control points. Base maps (8 nos.) were drawn on 1:2000 scale showing various features as such as roads, streets, culverts, bridges, Govt. Offices, Residential building etc. A composite map of Silvassa town on 1:5000 scale was also prepared.

9.3 DESIGN SCENARIO

9.3.1 Design Period

A design period of 30 years excluding construction period is adopted for design of the sewerage system as recommended by CPHEEO manual. Sewer pipes and sewage treatment plants are designed for design period of 30 years. For economical design, sewage treatment plants are initially designed for 5-10 years with provision for future expansion. Design period for pumping machineries is generally 15 years. Since project finalisation & approval by competent authority, land acquisition for pumping stations and treatment units, preparation and finalisation of tenders is likely to take around a year & allowing period of 3 years for construction of project, base year has been kept as year 2005.

9.3.2 Population Projection

As mentioned above, different components of the proposed system will be designed for 15 year/30 year design periods. Accordingly population of the town has been projected for base year of 2005 and also for intermediate stages of 2015/2020 and ultimate stage of 2035. The details are given below.

Population of Silvassa and Amli in the last four decades as per census records is as under.

POPULATION OF SILVASSA AND AMLI : 1961-91

Year	Silvassa	Amli	Total
1961	2646	3222	5868
1971	4495	4501	8996
1981	6914	6621	13535
1991	11725	12883	24608

Annual Population growth rates during the decades 1961-71 & 71-81 were 5.2% and 5.05% respectively which are almost the same. The population growth rate for the decade 1981-91 has abruptly risen to 8.1%. This unusual growth rate for population can be attributed to considerable industrial development and migration from within and outside the territory. Average annual growth rate during three decades i.e from 1961 to 1991 works out to 6.12 %.

The projected population estimates of Silvassa and Amli as assessed by different methods viz (a) Arithmetical increase method (b) Incremental increase method (c) Geometrical Increase method and (d) Semilog method are given in the following table.

POPULATION PROJECTION

Sl. No.	Method	2005	2015	2020	2035
1.	Arithmetic increase	33353	39600	42723	52093

Sl. No.	Method	2005	2015	2020	2035
2.	Geometric increase	47670	76449	96813	196616
3.	Incremental increase	40027	55808	65188	99287
4.	Semi Log	47000	75000	96000	200000
Regional Plan					
5.	(2000-2020)	55000	95000	120000	136000

Special incentives offered to establish industrial units in urban area from April 1993 have attracted fairly large number of small and medium scale units in the last few years. Industries create direct as well as indirect employment and, therefore, attract people from other regions specially the skilled labour. The population of the town is thus bound to increase substantially due to migration besides the natural increase. Assessing the future volume of industrial growth, employment potential and participation rate the Town & Country Planning Department has estimated the population in the year 2001, 2011 and 2021 as 42170, 72427 and 124668 respectively {Source : Regional Plan (2000-2020)}. The same data has been adopted as such in the project report. It has been presumed that population in the year 2021 shall almost be reaching the saturation point and accordingly further forecast for the year 2035 has been worked out on basis of Logistic Curve method. Population in the base year (2005), Intermediate years 2015 & 2020 and ultimate year (2035) thus works out to 55000, 95000, 120000 and 136000.

Out of total area of 1717.68 ha, as per land use plan of Silvassa town 421.46 ha area has been earmarked for residential development. Based on the above mentioned population projection for various stages, population densities in the project area for various stages are given below.

Intermediate Stage	(2015)	225 persons/ha.
Ultimate Stage	(2035)	323 persons/ha.

9.3.3 Per Capita Sewage Flow Rate

Silvassa water supply augmentation scheme is designed at rate of 140 lpcd. Water supplied to the community is assumed to be discharged in sewers @ of 80% as per CPHEEO manual. Thus sewage rate from community works out to 112 lpcd. Taking contribution of 3 lpcd from commercial and institutional bodies, sewage flow rate works out to 115 lpcd.

9.3.4 Ground Water Infiltration

As water table is at a depth of 3.0 m, there may be reaches where ground water table is not likely to be encountered, yet most of the reaches may have to be laid below ground water table. Besides, entry of water through leaking manholes or household connections have to be taken into consideration. To make allowances for such likely infiltration of ground water, infiltration of 5000 l/ha/day has been considered as per recommendation of CPHEEO manual. Ground water infiltration has been computed in accordance with the foregoing criteria.

9.3.5 Peaking Factors

To account for fluctuation in the rate of sewage flow following peaking factors have been adopted for different contributory population

Contributory Population		Peaking Factor
Upto 20,000		3.0
20,000 to 50,000	-	2.5
50,000 to 7,50,000	-	2.25

9.3.6 Self Cleansing Velocity

As the industrial wastes have not been included in the design of sewage, suspended solids are likely to be of smaller size and low specific gravity. Therefore, a minimum velocity of 0.35 m/sec is kept as self cleansing velocity, whereas in trunk sewer the velocity developed is 0.60 m/sec for present flow and 0.70 to 1.12 m/sec at designed peak flow. Wherever possible slopes for minimum velocity recommended by CPHEEO manual have been adopted. Links, which develop a velocity less than 0.35 m/s, facilities for flushing with help of mobile tankers, have been recommended.

9.4 PLANNING AND DESIGN OF SEWER NETWORK

9.4.1 Sewerage Zones

According to topography of town, entire project area has divided into six Zones (1 to 6) as shown in Fig. 4.1. Zone 1,3 and 4 have been subdivided into two sub zones each. As per statistics of proposed land uses, total residential area in the project area would be about 421.5 ha. From angle of implementation, project area has been divided into two part: Undeveloped and low lying areas of zone 3 where no buildings & roads exist and partly developed area of zone IV (zone 3A and 4B) have been proposed in Phase II and remaining area is proposed in Phase I. The implementation of Phase II project shall be taken-up as and when development in this area takes place. Zone and sub zone wise residential area covered in each zone is given below.

Zone	Sub- Zones	Residential Area (ha)
1	1A	61.92
	1B	19.04

Zone	Sub- Zones	Residential Area (ha)
2	-	63.14
3	3A	38.73
	3B	39.27
4	4A	56.87
	4B	49.55
5	-	51.06
6	-	41.88

Sample calculations of sewage flow for each of the sub-zone of zone-1A is given in Annex-1.

9.4.2 Layout of Sewer Network

Circular RCC sewers shall be laid underground in proper slope to collect the sewage from contributory population of sub zones and carry it under gravity to point of outfall. Sewers shall be laid on opposite side of the road carrying drinking water mains to avoid any contamination. The trunk sewers are proposed along main roads, branch sewers and laterals shall be laid along the branch and lateral roads respectively.

Total length of sewer line in the entire project area is 24.46 km. As the project area is very undulating, depth has been limited to 6 m. Intermediate pumping stations have to be provided to limit the depth of sewers and based on the topography of various zones. Nine pumping stations have been planned in the project area, with total length of rising main of 4650 m. Since subsoil water is at 3.0m depth it may not be practical to lay sewers in greater depth as pumping out of water round the clock in such reaches shall be difficult and nearby buildings are also to be protected. In laying of sewers normally 1.0 meter cover is provided to avoid any damage to sewer.

Manholes of branch and lateral sewer falling in each zone are numbered by simple arithmetical numbers i.e. 1,2,3,4, …… and so on. Various types of manholes have been provided to facilitate cleaning and inspection of sewers, depending upon depth of sewers. They are provided at each junction of sewers, change of grade, change in direction, at place of drops and at 30 m interval on straight runs. Rectangular manholes have been proposed upto depth of 0.9 m below ground level and beyond this depth circular brick manholes have been proposed. Drop manholes have been proposed to minimise turbulence where fall is more than 0.6 m. Total of 961 man holes have been provided.

For the purpose of network analysis by sewer software, entire sewer network has been transferred into a link-node network.

9.4.3 Design of sewers

Design of sewers has been done using 'SEWER' software. Manning's formula has been used for design of sewers. Various design parameters adopted for designing sewer network of various zones are given below:

Minimum velocity in sewers	:	0.6 m/s
Maximum velocity in sewers	:	1.1 m/s
Maximum d/D for sewer	:	0.80
Minimum allowable cover for sewers	:	1 m
Maximum allowable cover for sewers	:	6 m
Ground water infiltration rate	:	13.46%
Manning's coefficient (n)	:	0.013

Peak factors : In accordance with the serving population of the zone.

In addition to above, diameter progression of sewers has been considered for designing sewers. Drops have been provided wherever necessary.

Sewer links in some of the initial reaches have been designed manually to generate greater velocities. Flushing through mobile tankers shall be resorted to in the sewer links in which velocity is less than 0.60 m/s. In some of the reaches excavation depth is between 0.6 m to 1 m. In these reaches concrete encasement shall be provided over the sewer links to protect these from damage due to vehicular traffic. For sewer links which shall be laid below ground water table, concrete cradle bedding is recommended. NP2 RCC pipes have been proposed for sewers of size less than 500 mm. For sewer sizes above 500mm, NP3 pipes have been proposed. Sewer network design for zone-1A is given in Annex -2.

9.5 Design of Wet Wells

Circular shaped wells which are structurally stronger have been adopted. The storage capacity of the wells is designed considering the time for which the sewage is to be retained and frequency of pumping operation. Detention time is so kept that deposition of solids may be avoided and septic conditions may not develop. Maximum detention time is not to exceed thirty minutes of average flow. Design of wet well for the pumping station in zone-1A is given below

Design period	-	30 years
Average Flow (2020)	-	36.58 LPS
Average Flow (2035)	-	39.28 LPS
Peak Flow (2020)	-	100.38 LPS
Peak flow (2035)	-	109.1 LPS

Assume cycle of pumping operation = 6/hr (automatic operation)

$$\text{Volume of wet well (V)} = \frac{\text{K x Peak Flow (LPS)}}{6}$$

Where ,

V = Volume of wet well in cum.

Q =Peak flow in LPS

K = constant = 0.9

$$V = \frac{(0.9 \times 109.1)}{6}$$

V = 16.365 cum

keeping 1.0 metre effective depth of liquid in wet well

$$\text{Dia of well} = \sqrt{\frac{16.365 \times 4}{1 \times \pi}}$$

= 4.56 metre.

Considering the space required for installation of pumping machineries, dia of 6.0 m is adopted.

9.5.1 Structural Details Of Wet Wells

Civil structures of wet well have been designed as per I.S code of practice for circular tanks. Details of structural components and reinforcements of wet well in zone-1 is given in the enclosed drawing in Annex-6. Adequate quantity of waterproofing chemicals in concrete and cement plaster should be used as per instruction of manufacturers to ensure water tightness of the structures which are to be constructed underground and likely to encounter subsurface water. M20 concrete shall be used and a cover of 37 mm to the reinforcing mild steel plain bars should be provided in all the faces of concrete in contact with water. Base slab of all the wet wells are to rest on 75 to 150 mm thick lean M10 concrete. The layer of lean concrete should first be cured and then it should be covered with tarfelt in order to enable floor slab to act independent of the bottom layer of concrete. The cover slab of the wet wells are cast-in two semi-circular halves supported on central beam as shown in the drawings of wet wells. Lifting hooks should be provided in cover slab so that it could be removed for the purpose of installation of pumps or for their repair and maintenance. Ventilating shafts are also provided to keep the well ventilated properly. Control panels are provided on the top of the cover - slab housed in steel boxes as shown in the drawing of wet wells. Automatic operation with the help of electronic devices has been proposed for pumping station nos. 1,2,3,4,6,7 & 8. Semi -Circular pump chambers are provided on wet wells which are proposed to be operated manually. Submersible centrifugal sewage pumps of required capacity are provided in each wet well.

The details of arrangement of pumps, their suction and delivery pipes and fittings therein have also been shown in the enclosed drawing in Annex-6. Flanged M.S. pipes shall be used within the well and outside upto pumping mains sluice valve as shown in the drawings.

9.5.2 Design Of Pumping Machineries

Sewage submersible pumps coupled with electric motors are proposed to be installed in submerged condition under sewage in wet wells. Pumping stations with submersible pumps are economical as the dry pit for installation of pump and superstructure for protection of motor are not required. Besides, the problem of land acquisition are not encountered on account of underground installation of entire pumping machineries .

Submersible Sewage Pump

The pump should be of type and design suitable for pumping raw unscreened sewage with solid handling capacity of 80 to 150 mm dia.

Prime Mover

High Torque squirrel cage motors operating on 400/440 volts, 3 phase, 50 cycles, A.C. supply directly coupled on a common shaft with the pump are recommended, for installation in the wet wells designed for the purpose.

Materials for Construction of pump

Volute casing, impeller, Bottom plate, wearing, delivery pedestal should be of cast iron. Shaft should be of carbon steel and chrome plated in wetted portion. Guide rail pipe and chain should be of galvanized steel.

Computations of rating of pumps to be installed in Pumping station 1 is given below

Pumping Station no I (Zone - 1A)

Design period	-	15 years
Average flow (2020)	-	36.58 lps.
Peak flow (2020)	-	100.38 lps.
i) Static Head	-	8.48 m.
ii) Terminal Head	-	1.50 m.
iii) Frictional losses in pumping main	-	12.80 m.
iv) Losses in pipe fittings	-	1.28m.
		24.06 m.

Therefore design head (h) is adopted as 25 m.

Rated KW of pump at motor shaft taking 60% efficiency of pump

$$= \frac{1.02 \times 100.38 \times 25 \times 746}{76 \times 0.6 \times 1000}$$

$$= 41.88 \text{ KW.}$$

Provide 3 pump sets, 1 of 42 KW & 2 of 15.5 KW capacities capable of delivering 100.38 lps and 36.58 lps of sewage against a head of 25 m.

Sample design of pumping main in zone-1A is given in Annex-3.

9.6 PROVISION OF MANHOLES

Rectangular Manholes

Rectangular manholes of size 900 x 1200 mm have been provided on sewers laid 0.9m below G.L. as per recommendation of CPHEEO manual.

Circular Manholes

Circular manholes having internal diameter of 900 mm have been provided for depths above 0.9m and upto 1.65m, 1200mm for depths above 1.65m upto 2.30m and 1500mm for depths above 2.30m upto 9.0m.

All the manholes mentioned above have to be constructed in brick masonry, slanting in top portion, in order to have an opening equal to external diameter of the manhole cover. R.C.C. frame and cover have been proposed to avoid the risk of theft, which generally occurs in case of C.I. manhole frame and cover on account of their high scrap value. The R.C.C. manhole frame and cover shall also be economical. C.I. steps have been provided in each manhole for easy descent of maintenance personnel.

R.C.C. Manholes

R.C.C. Circular manholes of internal diameter of 1500mm for depth above 4.3 m upto 7.3 m are proposed, where high subsoil water conditions are encountered.

Drop Manholes

Vertical drop pipes are provided wherever level difference between peak flow level of main line and invert level of the branch line exceeds 600 mm. The vertical drop pipe is supported on W.I. brackets inside the shaft.

Structural details of various type of manholes are given in Annex 6.

MANHOLES IN VARIOUS ZONES

Details of various types of manholes in zone 1 are given in Annex 4. Summary of manholes provided in various zones is given below

Sl. No.	Zone	Numbers			
		Rectangular	Circular	Drop	Total
1	1	31	215	12	258
2	2	19	287	10	316
3	3	2	77	4	83
4	4	9	112	8	129
5	2	2	110	5	117
6	3	3	53	2	58
TOTAL		**66**	**854**	**41**	**961**

9.7 DESIGN OF STP

Conventional process of sewage treatment like activated sludge process, trickling filters etc. have not been found conducive to the requirement of the town, in view of high cost and sophisticated operability of the treatment plant. Availability of large spaces and huge tracts of land, in and around the town, warrants the adoption of facultative aerated lagoons, which are commonly used due to their simplicity in operation, and minimum need of machineries. Therefore, aerated lagoons have been proposed for treatment of domestic sewage. Lagoons may be used in parallel or in series to achieve special objectives. Series operation is beneficial where a high level of B.O.D removal or coliform removal is important. The effluent from lagoons in series operation has a much lower algal concentration than that obtained in parallel operation with resultant decrease in colour and turbidity. After facultative lagoons sedimentation pond may be desirable which also serves as maturation pond for effluent. Land has been earmarked along the proposed ring road (north , west side) for construction of sewage treatment units.

Following factors have been considered in process design of proposed aerated lagoons.

(a) B.O.D. removal
(b) Effluent characteristic
(c) Oxygen requirement
(d) Power requirement for mixing
(e) Solid Separation

Sewage Treatment Plant is generally designed for a design period of 30 years. Keeping in view economy the Plant is initially designed for design periods of 5-10 years. Provision is kept for future expansion.

Based on the network analysis, average and peak sewage flows for various stages are given below:

	Average flow	**Peak flow**
Base Flow (2005)	73.53 lps	205.55 lps
Intermediate stage (2015)	150.84 lps	397.81 lps
Intermediate stage (2020)	181.27 lps	493.32 lps
Ultimate stage (2035)	205.44 lps	559.11 lps

The volume of lagoon is calculated based on the average flows. It is seen that there is very small increase in average flows from year 2020 to 2035 as the population is expected to reach saturation point in year 2020. In view of this, it is proposed to design the lagoon initially for year 2015, keeping provision for expansion based on average flows in year 2035.

Computations for Year 2015

BOD_5 of raw sewage - 250 mg/l (assumed)

Winter Temperature - 15^o

Summer Temperature - 35^o

Sewage flow = 13033 m^3/day

Assume 5-day detention time

Volume of lagoon = 13033 x 5 =65165 m^3

Keeping 4.0 m depth of liquid in the lagoon

Surface Area = $\frac{65165}{4}$ = 16291.25 m^2

Provide 4 units of lagoon of size 24 m x 170 m, 2 in series & 2 in parallel with baffles at 8m centre to centre.

Computations for Ultimate - Stage (2035)

Additional lagoon volume required

= 4717.44 x 5

= 23587 m^3

= 23587/ 4 = 5897m^2

Provide 2 units of lagoon of size 18m x 170m in parallel to the existing system after expiry of year 2015. Provide one of baffle in middle to achieve plug flow regime.

B.O.D.$_5$ Removal

For facultative aerated lagoons substrate removal rate (K) for sewage varies from 0.6 – 0.8 per day (soluble BOD basis) at 20^{o}C. At other temperatures in lagoon the value is given by formula.

$$K_T{}^o{}_C = K_{20}\,c \text{ X } (1.035)^{T-20}$$

The temperature in lagoon T_L is estimated from the following equation

$$\frac{Dt}{h} = \frac{(T_i - T_L)}{f\,(T_L - T_a)}$$

Where

Dt = detention time

h = depth of lagoon

T_I = T^{o}C of influent waste water

T_a = T^{o}C of ambient air

T_L = T^{o}C of lagoon

f = heat transfer coefficient taken as 0.49 m/day

Ambient winter air temperature – 15^oC. Assuring influent sewage temperature as $20^o C$ and applying following equation.

$$\frac{Dt}{h} = \frac{(T_i - T_L)}{0.49\ (T_L - T_a)}$$

$$\frac{5}{4} = \frac{20 - T_L}{0.49\ (T_{L -}\ 15)}$$

$2.45\ T_L - 36.75 = 80-4T_L$

$6.45\ T_L = 80+36.75$

$T_L = \frac{116.75}{6.45}$

$T_L = 18.10^oC$

<u>Estimation of K</u>

Assume $K_{20^oC} = 0.7$/day

$K_{18.10^0} C = 0.7\ (1.035)^{18.10}$

$=0.7\ (1.035)^{-1.9}$

$= 0.656$

Lagoon geometry has been kept in such a manner that flow condition shall develop in lagoon of plug flow type and under these conditions D/uL = 0.2 approx.

Where

D = Axial dispersion coefficient (length/time)

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

L = Length of axial travel paths

BOD_5 removal efficiency (in winter)

KD_t = 0.656 x 5 = 3.28

For values of KD_t = 3.28 and D/uL = 0.2

Soluble BOD_5 removal efficiency = 91%

Therefore soluble BOD_5 in effluent = 22.5 mg/l

S.S. likely to flow out in effluent = 40 mg/l

BOD of V.S.S. = 0.77 (.6 x 40) mg/l

= 18.4 mg/l

Hence BOD in effluent = 22.5+18.4 = 40.9 mg/l

Overall efficiency 83.64%

In other months of the year the efficiency will be higher due to rise in temperature and effluent BOD will be less than the above value

Oxygen Demand

The amount of oxygen required has been found to vary from 0.7 to 1.4 times the amount of BOD removed.

When efficiency = 83.64.% and all BOD is removed aerobically

O_2 required/day = 0.836 (1.4x3258.25)

= 3813.45 kg/day

= 158.89 kg/hr

Power Requirement

Assuming O_2 = 2 Kg/KWH for power delivered at shaft.

Power needed $= \frac{158.89 \text{ Kg/hr}}{0.8 \times 2 \text{ Kg } O_2 \text{/kWh}}$

= 99.30 KW

= 133.10 H.P. say 134 H.P

Provide 24 nos. of high speed floating surface aerators of 6 H.P. capacity i.e. 6 nos. for each lagoon suitably spaced.

Power level in lagoon $= \frac{99.30 \text{ KW} \times 1000}{65165}$

= 1.52Watt/Cum <1.8 W/Cum

Hence acceptable

Solid Separation

The effluents from aerated lagoons contain high concentration of suspended solids which should be necessarily settled down to bring down the BOD load of effluent within prescribed standard of regulatory agencies prior to discharging in streams. Therefore, a polishing pond or maturation pond has been provided as a secondary treatment.

Detention time = 1 day

Pond volume = 13033 m^3

Taking pond depth = 1.8 m

Surface Area $= \frac{13033}{1.8} = 7240.5 \text{ m}^2$

Keeping width of the pond 60.00 meter

$L = \frac{7240.5}{60} = 120.67\text{m}$

Adopt 60 m x 121.0 m pond size

Site plan of STP site on the topographical map, layout of various units of STP alongwith inlet and outlet arrangement, piping and necessary fittings etc. are shown in the drawing given in Annex -3.

Design of Bar Racks and Screen Chamber

Hand cleaned bar racks are provided as designed below,

Assuming velocity of flow through bar racks = 1 m/sec

Peak flow = 0.56m^3 /sec

Clear area required = $\frac{0.56}{1}$ = 0.56 sq.m.

Taking depth of flow in screen chamber 0.50 m

Clear width of the channel = $\frac{0.56}{0.50}$ = 1.12 m

Clear opening between bars = 25 mm

Width of bars 10 mm

(Bars of rectangular section 10 mm x 50 mm, parallel to direction of flow)

No. of openings = $\frac{1.12}{0.025}$ = 44.80

Provide 45 openings of 25 mm width and 44 bars of 10 mm wide.

Actual width of screen chamber = 45 x 25 + 44 x 10

= 1155mm + 440 mm = 1565 mm

= 1.565 m

Measuring device : 90^{O} triangular notch has been provided following screen chamber to measure the flow of liquid as shown in drawings annexed.

Bar rack shall be kept 45^{o} inclined to horizontal.

Land requirement

Land requirement for different components of sewage treatment plant such as screening chamber, lagoon, maturation pond (including future units) and main pumping station works out to 9.5 ha.

Net land requirement	:	8.5 ha
Total land requirement	:	9.5 ha

9.8 COST ESTIMATES

The cost estimate of different components of the project is based on SOR (Schedule of Rates) of GWS&SB (Gujarat Water supply & sewerage Board) which is followed by Territory Administration. The items which are not covered by the said S.O.R are worked out at prevailing market rates like cost of sewage submersible pumps, surface aerators etc.

A lump sum amount for house connection is provided. Similarly lump sum provision has been made for staff quarters, land acquisition, laboratory building & required equipment like glassware, chemical, reagents etc. Three year time has been provided for implementation of the project which may extend on account of unavoidable circumstances in course of execution of work. Therefore a price contingency of 15% and physical contingency of 10% over total cost of project have been included in the final estimate of project cost .

Annual operation & maintenance expenditure on the project has also been calculated in accordance with the standard yard stick of maintenance staff and management personnel and necessary provision for spares of pumping machinery, electrical equipments tools etc. have been made in the estimate. It also includes maintenance cost of civil structures, mechanical parts of water treatment plant, cost of annual energy charges for running of pumps, aerators etc.

Maturation pond may be used for pisiculture by growing duckweed plant in pond which may lighten the burden of cost recovery. Annual Operation & Maintenance (O&M) cost works out to Rs 45.9 lakhs. Sample calculations of cost of zone-1A for Sewer Collecting System, Pumping Stations, Pumping Mains and Sewage treatment plant are given in Annex-5. Abstract of the Cost of the scheme is given in the following Table

COST ABSTRACT

COST COMPONENT	ESTIMATED COST (Rs. Lakhs)
Sewer Collecting System	217.41
Pumping Stations	11.51
Pumping Mains	74.84
Pumping Machinery	94.52
Sewage Treatment Plant	194.70
Power Supply	8.74
Staff Quarters (L.S.)	10
House Connection (L.S.)	50
Land Acquisition (L.S.)	50
Lab. Building & Testing Equipment (L.S.)	5

COST COMPONENT	ESTIMATED COST (Rs. Lakhs)
Vehicles (L.S.)	10
ETP charges	54.35
Sub-Total	**781.07**
Price Contingencies @ 15%	117.16
Sub Total	898.23
Physical Contingencies @ 10%	89.82
Grand Total	**988.05**