5.4 NRW REDUCTION PLANNING

5.4.1 NRW Reduction Strategy Development

PHE aims to progressively implement enhanced services. Where this includes the provision of 24 hour supply systems, PHE will need to ensure that water supply networks are managed effectively to maintain NRW at an economic level. It is well proven that a focus on reducing NRW will produce a positive financial return based upon operational savings and capital deferment.

The aim of a NRW Reduction Strategy will be to:

- Maximise the use of available water resources
- Improve the efficiency of water supply systems
- Improve services to customers
- Defer capital investment
- Reduce operating costs through water savings
- Increase revenue through water savings

(1) Control Philosophy

To ensure that NRW is maintained within acceptable limits, PHE will need to introduce NRW control methods that facilitate the monitoring of NRW levels within discrete zones – 'district metering areas' (DMA's). In this way, DMA's with unacceptable levels of NRW will trigger NRW reduction activities to be undertaken. Establishing DMA's will enable continual monitoring of NRW within each DMA by the use of 'total net flow' and 'minimum night flow' methods (for 24 hour supply systems). The 'total net flow' method is used to determine the total NRW within each DMA and the 'minimum night flow' measurement is used to determine leakage levels. Continual monitoring of flows and NRW within each zone will ensure that the appropriate measures are taken to ensure that NRW remains within acceptable limits. Over time, the level of NRW will increase due to asset deterioration and eventually an unacceptable level of NRW will be reached, thus triggering a renewed cycle of NRW reduction activities. The unacceptable level of NRW is determined through a simple cost benefit analyses, whereby the cost associated in reducing NRW is compared to the savings made.

(2) Implementation

Implementing a NRW Reduction Strategy involves the application of standard management techniques and processes that are considered the norm for any well run water undertaking.

A successful NRW Reduction Strategy will require:

- **Leadership** – from the top of the organisation, there must be a "Champion" to ensure that the whole organisation concentrates upon the basics of increasing income and reducing the physical leakage.
- **Commitment** – throughout the organisation there must be a determination to follow through the processes that reduce NRW.
- **Resources** – significant resources are required to make the step change necessary to reduce NRW. Once NRW is under control and efficient and effective processes are in place then the resource can be reduced to a lower level. It must be recognised that NRW control is an ongoing operation.

In order to implement a successful NRW reduction strategy, PHE will need to:

- Get the basics right now to control and reduce the current levels of NRW such as capturing accurate data required to monitor and control physical and commercial losses
- Implement 'Active Leakage' control techniques to reduce the current levels of UFW
- Develop staff and systems for progressive and sustained improvements in NRW
- Minimise future leakage by raising standards of installation and repair
- Minimise future commercial losses by raising standards of metering, billing and revenue collection
- Undertake 'enabling works' to monitor and control UFW in future. PHE will need to consider contracting out the enabling works and the 'primary UFW reduction' to an agreed target level. Example; reduce UFW from it's current level of 35% to 28-30% in 5 years through the setting up of DMA's and conducting active leakage techniques. Following this period, PHE would need to take responsibility for ongoing UFW control

5.4.2 Current Status

Currently, PHE operate a "Passive" strategy whereby leak detections and repairs are managed on a reactive basis such that only visible leaks are dealt with. Due to low pressures it is likely that many leaks will not appear above ground and therefore go unnoticed. Many of the leaks will be as a result of poor materials, installations or repairs. Leaks can cause water quality issues due to back-siphonage as well as causing commercial losses.

In relation to the four basic leakage management activities considered to be best practice, PHE's current approach is to:

- Pressure and Flow Management
    - Throttle valves due to lack of pressure reducing valve (PRV) set-up

    - Distribute water based on estimated capacities, pressures and flows
- Speed and Quality of Repairs
    - Allow leaks to go unmonitored
    - Allow repairs to go unmonitored
    - Allow the use of poor quality materials and workmanship
    - Allow temporary repairs due to lack of materials (these become permanent repairs or need revisiting)
- Infrastructure Management
    - Replace/rehabilitate pipes in a limited and adhoc manner
    - Operate 'open network' systems with little monitoring or control
    - Operate the networks without discrete zones (DMA's)
    - Allow the installation of inferior connections and meters
- Active Leakage
    - Operate a 'passive' approach to managing leakage
    - Operate with limited equipment
    - Carry out leakage work during the day unless emergencies dictate night working

Based on the above and a review of current NRW practices PHE will need to consider the following strategic initiatives to enable the effective reduction and control of the current level of NRW in future:

- Allocate responsibilities to a 'process owner' (champion) and individual managers for NRW reduction within each Region
- Prioritise NRW reduction in high marginal cost areas or areas where water shortfalls are the highest
- Introduce digital mapping and map all existing supply networks
- Introduce Network Analysis across Goa
- Reduce excess pressures in the networks
- Implement an Asset Management Plan; example, replace 2% of the network/year
- Improve the control and quality of network extensions
- Introduce a process for the accreditation of contractors to improve quality of work
- Develop and introduce Key Performance Indicators (KPI's) to improve management control

(1) Analysis of UFW and NRW

As can be seen from the figures below, it is difficult to measure NRW and UFW accurately, however, due to a high degree of correlation between the different methods and studies, the figures shown can be used as a good basis for developing a reduction strategy.

Based on the sector status study (August 2004):

- UFW (Real and Apparent losses) was calculated to be 34%
- NRW (Real and Apparent losses and authorised unbilled consumption) was calculated to be 47 % as water supplied via stand posts was estimated to account for 13% of consumption
- 17% of domestic, 16% of commercial and 26% of industrial meters don't work
- A high proportion of working meters are inaccurate
- 15% of treated water is lost during transmission from treatment plants to service reservoirs (real losses + unauthorised consumption)
- 22% of water is lost in distribution from reservoirs to customer taps (real + apparent losses)

Based on the leakage survey conducted by the JICA Study Team during the first phase study

- NRW (Real and Apparent losses and authorised unbilled consumption) was calculated to be 49 % (Ranged from 13 to 74%)
- 15-58% of meters do not work
- A high proportion of working meters are inaccurate
- 40% of leaks detected were on distribution lines, 60% were on supply connections

Based on the 'NRW reduction pilot project' conducted by the joint PWD/JICA Study Team during the feasibility phase, April/May 2006

- NRW (Real and Apparent losses and authorised unbilled consumption) was calculated to be 57% prior to active leakage control measures
- NRW was reduced to 49% in the pilot area as a result of the pilot study
- Approximately 50% of meters don't work or are not readable
- 50% of leaks detected were on distribution lines, 50% were on supply connections
- A number of supply connections were not billed
- A number of illegal connections were identified

The NRW reduction pilot project has been invaluable in satisfying the following objectives:

- Reduction of NRW through accurate determination of NRW ratios for the pilot area and use of practical measures to reduce the key elements
- Demonstrating the benefits of an 'active' leakage approach in reducing the levels of NRW in the pilot area through adoption of leakage mitigation measures
- Providing data, experience and knowledge for the development and implementation of a state wide NRW reduction and control program

- Providing experience and data for the development of interim/short-term operational improvement scenarios that could potentially yield significant results in terms of NRW reduction
- The transfer knowledge and technology through training and active participation of PWD staff in the pilot project
- Building enthusiasm at senior level for adopting an 'active' approach to NRW management and for PWD to initiate the development of a 'NRW reduction roll-out programme' across the state

Based on the 2004-5 financial analysis conducted by the JICA Study Team during the first phase study

- NRW was calculated to be 50.6%
- Unit selling price of water was calculated to be Rs.8.66 per m^3 billed
- Unit cost of producing water was calculated to be Rs.12.38 per m^3 billed

Based on the above unit costs and assuming the volume of water into supply of 341MLD (from Sector Status Study report 2004), the following cost savings or increased revenues could be achieved depending on the level of UFW reduction activity to reduce UFW from the current level of 34% to the levels indicated:

Table 54.1 Potential Benefits in Reducing UFW

Potential cost saving (Million Rs.) If UfW reduced to:			Potential increase in revenue (Million Rs.) If UFW reduced to:		
30%	25%	22%	30%	25%	22%
61.635	138.679	184.905	43.115	97.008	129.344

(2) Conclusions

The distribution system is the most difficult asset to manage due to the fact that it is unseen and requires a higher level of management focus that it currently receives if NRW and UFW ratios are to be reduced effectively.

Cost savings and/or increased revenues will result from increased efforts to reduce the current levels of leakage and NRW.

It is evident that there is a lack of comprehensive systems necessary to manage, operate, develop and monitor the existing distribution networks. 'Enabling works' will be needed to rectify this.

Although there is a considerable amount of discussion about NRW within PHE, there is limited action to bring NRW under control and within economic limits. Apart from the limited reporting being carried out and the projects currently underway to focus efforts, the current approach can be considered ‘passive’ at best. Having said that, following the success of the recent NRW reduction pilot project, the PWD Secretary has instructed (NRW pilot project workshop 01 June 2006) that PWD ‘forge ahead’ with a ‘NRW reduction roll-out programme’ across the state and suggested that this be led by PWD, prepared by the JICA Study Team.

(3) Way Forward

Based on the review of current NRW practices and experience gained from the pilot studies, in considering a NRW Reduction Strategy for the future PWD will need to consider implementing the following key initiatives:

- Improve network management practices
- Agree standards for new connections and repairs including standard specifications for materials, fittings, meters, layout, non-return valves, sealing, testing, calibration etc
- Introduce leakage policy and improved methods
- Replace all defective (leaking) house connections
- Repair all existing visible leaks
- Introduce metering policy and improved practices
- Replace all defective meters
- Conduct enabling works and leakage control measures
- Set up Active Leakage Teams within each Division or Region with appropriate tools to find and fix leaks
- Institutionalise NRW management measures and tackle ‘apparent’ as well as ‘real’ losses
- Ensure 100% billing and improve revenue collection practices
- Build on experience gained from the pilot NRW reduction programmes
- Roll-out NRW reduction mitigation measures across Goa. Refer to Volume III Chapter 4 NRW Reduction Roll-out Plan

5.4.3 NRW Reduction Planning

In order to put together an effective strategy for NRW reduction it is essential to understand the components that make up NRW in order that each element can be understood, measured, monitored and controlled. Based on this premise, the following terminology and reduction planning principles have been 'shared' with the NRW Reduction Pilot team and put to good use in conducting the pilot study. These same principles have been adopted in the development of methodologies that will be used in the NRW Reduction Roll-Out Programme. Refer to Volume III Chapter 4 NRW Reduction Roll-out Plan.

(1) Terminology

With the increasing emphasis on sustainability, economic efficiency and care for the environment, the problem of water losses from water supply systems is of major interest worldwide. However, difficulties can arise in calculating, defining and comparing losses due to varying use and interpretation of definitions and terminology. For purposes of the PHE Master Plan, the International Water Association guidelines have been used as follows:

Water Balance

The components of 'water balance' for a transmission or distribution systems are as follows:

<table>
<tr><td rowspan="9">System Input Volume m^3/year</td><td rowspan="4">Authorized Consumption m^3/year</td><td rowspan="2">Billed authorized consumption m^3/year</td><td>Billed Metered Consumption</td><td rowspan="2">Revenue Water m^3/year</td></tr>
<tr><td>Billed Unmetered Consumption</td></tr>
<tr><td rowspan="2">Unbilled authorized consumption m^3/year</td><td>Unbilled Metered consumption</td><td rowspan="7">Non-Revenue Water m^3/year</td></tr>
<tr><td>Unbilled Unmetered consumption (such as public taps)</td></tr>
<tr><td rowspan="5">Water Losses m^3/year
UFW</td><td rowspan="2">Apparent Losses m^3/year</td><td>Unauthorized Consumption</td></tr>
<tr><td>Metering Inaccuracies</td></tr>
<tr><td rowspan="3">Real Losses m^3/year</td><td>Leakage on Transmission & Distribution Mains</td></tr>
<tr><td>Leakage and Overflows at Utility's Storage Tanks</td></tr>
<tr><td>Leakage on Service Connections up to point of Customer metering</td></tr>
</table>

Source: IWA October 2000 "Losses from Water Supply Schemes: Standard Terminology and Recommended Performance Measures"

Figure 54.1 Components of Water Balance

System Input Volume

The 'System Input Volume' is the volume of water input to the system.

Authorised Consumption

'Authorised Consumption' is the volume of water taken by authorised registered users, including the utility and others who are implicitly or explicitly authorised to do so. As well as public taps, this will include fire fighting, flushing, street cleaning, watering of parks, building construction etc. whether metered or not.

Water Losses

'Water Losses' is calculated as 'System Input Volume' – 'Authorised Consumption'. 'Water Losses' consists of 'Real' and 'Apparent' losses and equates to Unaccounted for Water (UFW).

'Real Losses' consists of physical water losses from a pressurised system up to the point of customer connection. The volume lost through leaks, burst and overflows will depend on frequencies, flow rates and duration of leakage. Physical losses consist of actual, uncontrolled escape of water that enters the ground and is lost from the system. Normally these occur from trunk mains, service reservoirs, water towers and from distribution mains and services up to the customers' connection. This resource of water could be sold at full price where there is a shortfall of demand.

Over-registering of source meters (or over-estimation of water into supply) and under-registering of customer meters (or under-assessment of consumption when meters don't work) leads to over-estimation of 'Real Losses'. The opposite affect of these scenarios leads to under-estimation of 'Real Losses'.

'Apparent Losses' consist of 'unauthorised consumption' (theft or illegal use) and inaccuracies, associated with source and customer metering.

Non Revenue Water (NRW)

'NRW' is defined as the difference between the 'System Input Volume' and 'Billed Authorised Consumption'. This is equivalent to the sum of 'Unbilled Authorised Consumption', 'Apparent Losses' and 'Real Losses'.

Unaccounted for Water (UFW)

'UFW' is defined as the proportion of water entering a system, whose final destination is unknown. This is equivalent to the sum of 'Real' and 'Apparent Losses' or NRW less 'Unbilled Authorised Consumption'.

(2) Demand Forecasting

A demand forecast has been prepared for the Master Plan. For purposes of the demand forecast, UFW is assumed to decrease from 35% (currently level) to 21.7% (assumed level at 2025) over the life of the Master Plan. This will present opportunities for deferment of capital investment as illustrated below:

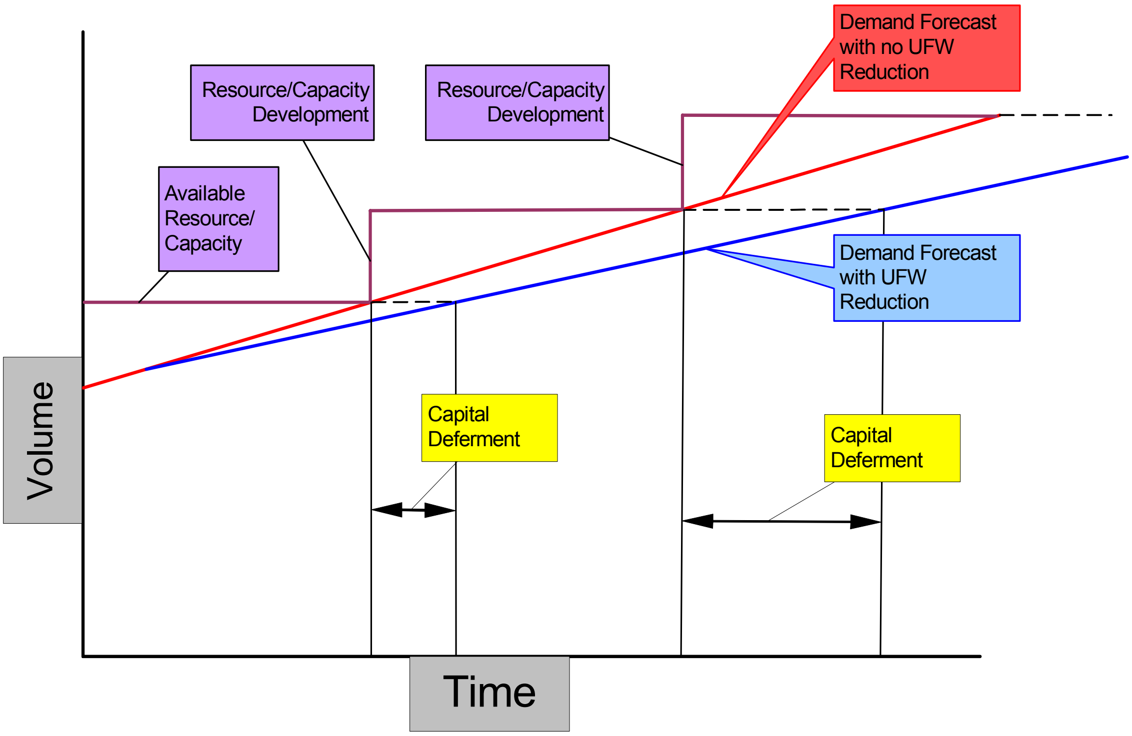


Figure 54.2 Demand Forecast Model

There are advantages and disadvantages in adopting a resource development solution to a water shortage as indicated below, however, in the majority of cases the II. NRW REDUCTION STRATEGY would be the preferred option:

Table 54.2 Resource Development Options

I. SOURCE DEVELOPMENT STRATEGY	
Advantages	Disadvantages
More customers get water Builds another asset for PHE Additional water relatively quickly Development of the resource is easier to manage	Increases total cost for water supply UFW remains at 35% Need to reinforce existing distribution system to cope with extra flow Increased pressure will lead to more leakage

II. NRW REDUCTION STRATEGY	
Advantages	Disadvantages
Saves valuable and limited water resources More customers get water Reduces total cost of water supply Defers the need for new plant Will improve PWD image Is environmentally friendly	Requires equipment and training Specialist consultants/contractors may be required Continuous process – long term liability

(3) Economics of UFW Reduction

A simplistic model of the economic level of leakage indicates that the cost of water lost is linear and the cost of leakage management is non-linear. The graph is affected by numerous variables including:

- Condition of the underground asset
- Pressures
- Cost of repairs/pipe replacement
- Cost of 'active' leak detection
- Availability of records and drawings

Currently, PHE's UFW stance is 'passive' and therefore expenditure on controlling UFW is minimal with a resulting high level of UFW. For illustrative purpose only, the model below indicates that PWD is operating uneconomically and also shows that leakage will never be zero.

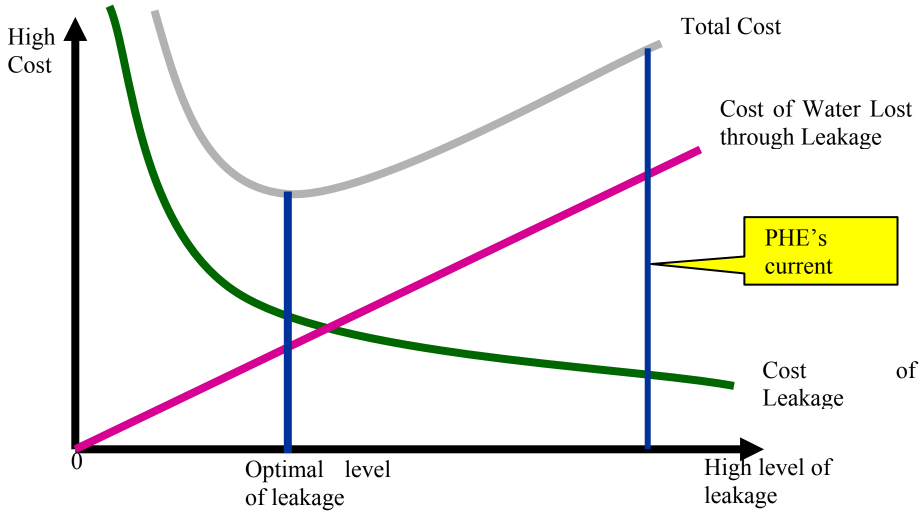


Figure 54.3 Economic Level of Leakage Model

(4) Leakage Management Principles

In order to address leakage, it is important to understand the key elements that influence leakage. This is particularly important when considering the economics of leakage management in order to allocate the appropriate level of resources to tackle the various leakage reduction approaches. The four main components of leakage management can be illustrated as follows:

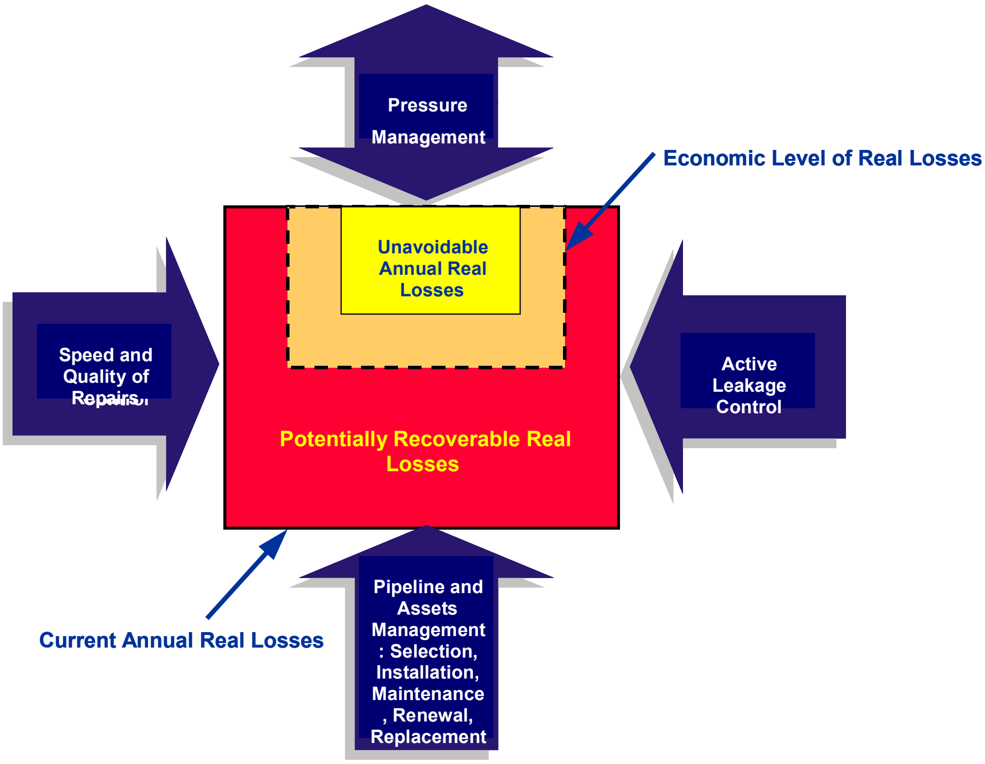


Figure 54.4 Main Factors Influencing the Level of Leakage

As can be seen, system leakage is highly dependent on several key factors. Lack of attention to any of these factors is likely to increase leakage and therefore it will be necessary to address all four issues simultaneously if leakage is to be reduced and maintained under control.

Each of these components is described in more detail as follows:

(a) Infrastructure Management

Infrastructure Management involves regular maintenance, repair and renewal of defective infrastructure including:

- Water Towers
- Reservoirs/float valves
- Transmission Mains and ancillaries
- Distribution Networks and ancillaries

- Service Connections
- Customer Meters

PHE will need to implement an infrastructure renewals policy to maintain current as well as future asset condition. Typically, renewals would rarely exceed 1 to 2% per annum and it is well proven that a well-targeted rehabilitation policy will have a positive impact on leakage. It is essential that the replaced mains and services are constructed to the highest standard so that the underground asset will provide good service for at least 50 years.

(b) Speed and Quality of Repairs

The volume of water lost through leaks is influenced by a number of factors including, pressures, flows, speed and quality of repair. It is important that good quality permanent repairs are made first time to avoid the need for repeat visits. Repairs should be undertaken as a high priority, usually the same day, and should be recorded by a monitoring system to ensure effectiveness. This includes the need to attend to leaking glands from valves and hydrants which are relatively easy to repair without excavation. The introduction of performance targets and measure is necessary to ensure a process of performance improvement. PHE should aim to repair 80% of all leaks within 24 hours and all leaks within 5 days. This enables very minor leaks to be postponed and difficult leak repairs to be planned.

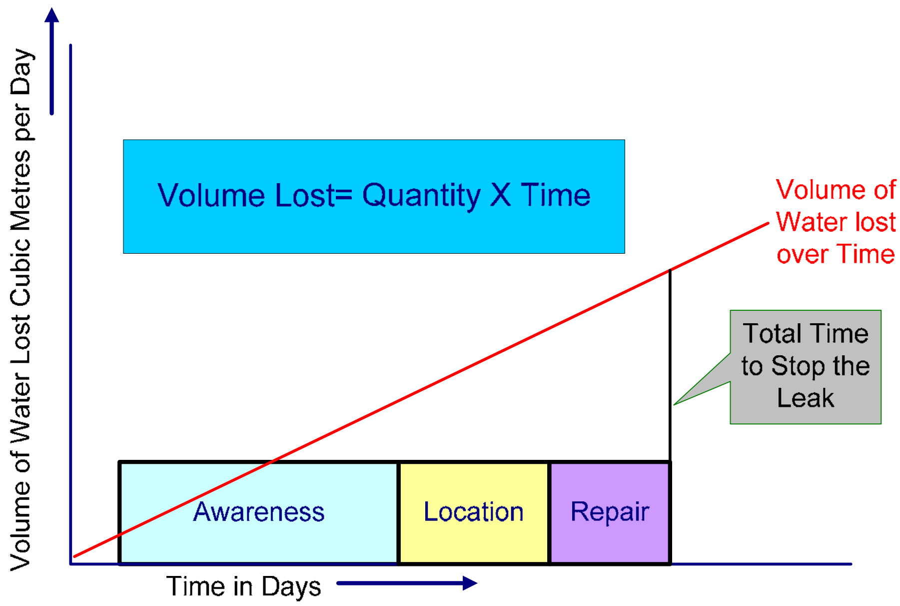


Figure 54.5 Influence of Speed of Repair on Water Losses

Considering the main components of the above chart:

The factors that affect 'awareness' are:

- District metering
- 'Step testing'
- Telemetry
- Regular 'sounding'
- Equipment availability
- Manpower availability
- Cooperation of the public and customers – in promptly reporting leaks

The factors that affect 'location' are:

- Knowledge of the network
- 'Step testing'
- 'Sounding'
- 'Leak Noise Correlation'
- Equipment availability
- Manpower availability

The factors that affect 'repair' are:

- Material quality and availability
- Accessibility of the leak
- Manpower availability
- Equipment availability

(c) Pressure Management

Pressure management is not extensively used by PHE at present.

Pressure management techniques are necessary to:

- Maintain pressures as low as possible whilst meeting the needs of customers in terms of service delivery. This saves energy and reduces 'stresses' on the network. PHE will need to install pressure reducing valves for this purpose
- Avoid excessive losses through leaks or the unnecessary creation of leaks caused by high pressures
- Create a stable network. Minimising interventions such as valve operation will reduce cyclical pressures and water surges

(d) Active Leakage Control

'Active' and 'Passive' leakage control are the terms used to describe proactive and reactive leakage control methods.

'Active Leakage Control' requires proactively sending leak detection and repair teams into areas to search for and repair unreported leaks.

'Passive Leakage Control' follows the passive approach of attending to visible leaks only and/or waiting for leaks to be reported before leak repair teams are sent to locate and repair reported leaks. This approach can result in many unreported bursts running for many months if not years, before they grow to such an extent that they are finally reported. The adoption of one strategy against the other will be influenced by the cost of repair compared to achievable savings resulting from leakage reduction.

It is well proven that a focus on reducing UFW to economic levels is cost effective in terms of savings in operational costs or increased revenues. PHE should change their current approach and implement an 'Active Leakage' Strategy to maximise the availability of water to its customers at least

District Meter Areas

District Meter Areas (DMA's) are an essential tool in the management of physical leakage in 24 hour supply systems. They provide a rapid method of determining increased leakage as well as assisting in the location of the leaks.

District metering is a process of dividing up a large open system into a series of smaller and discreet areas, using closed boundary valves, whose inlet supplies are permanently metered. These areas enable operations staff to gain a greater understanding of consumer demands, area pressures, daily/night flow patterns etc. More importantly, the UFW and leakage levels within each DMA can be calculated, allowing the operations staff to identify which areas within the zone have the highest UFW thus enabling prioritisation of leakage detection and repair activities. The permanently metered DMA inlet and outlets will enable a study of the 24hr demand profile within the DMA and in particular the measurement of minimum flow during the 24hr period which tends to be at night when most domestic customers are asleep and commercial & industrial customers are closed. Because of the limited demand during this minimum night flow (MNF) period all of the components of UFW, apart from leakage, are minimised. Therefore by studying MNF PHE will be able to calculate leakage within each DMA and prioritise their leakage reduction activities accordingly.

Source Metering

One of the most important aspects of UfW management is to know with a degree of certainty how much water is put into supply. Therefore it is vital that appropriate and accurate flow meters are installed on the outlets of treatment works. These should be calibrated regularly to ensure accuracy and reliability and should be compatible with other control systems that may be used in future such as telemetry, SCADA etc.

Telemetry

Future development of DMA measurement would be to telemeter the information back to a central point. This central point could be the divisional office, regional office or headquarters. Telemetry will provide early warning of major and minor leaks that occur and will reduce the response time to find and fix the leaks. In developing countries where leakage is an issue, telemetry systems are becoming commonplace and have been standard practice in developed countries for many years. Should PHE go ahead with the 24 hour DMA approach, they will need to consider telemetry for medium/long term planning.

(5) Commercial Losses

A NRW reduction strategy should address commercial losses as well as physical losses. Commercial losses are sometimes referred to as non-physical losses. This is because the water is not lost but it does not generate income and is not accounted for. There is reason to suspect that commercial losses are being experienced by PHE in a number of ways excluding the policy of 'authorised unbilled consumption' of street taps.

Commercial losses can occur as the result of:

- Meter fraud
    - Meter tampering
    - Meter bypassing
    - Meter reversal
- Free use of water (metered or un-metered) for
    - Official bodies
    - Fire fighting
    - Municipal activities such as street cleaning, watering parks and gardens
    - Water utility operations mains cleaning mains repairs
    - Theft

- Illegal connections
    - Illegal abstraction from the network, fittings, tankers, etc
- Administrative procedures
    - Non registration of connections
    - Meter reading errors (deliberate or otherwise)
    - Incorrect classification of commercial or industrial connections

The only way of detecting any of the above situations is to physically inspect installations on a regular basis. This should form an integral part of the NRW reduction strategy.

(a) Meter Reading

Meter reading affects PHE profitability and should therefore be considered as one of the most important functions to get right. The main purpose of metering is to generate income but the accuracy of the information or reading is also important in terms of assessing leakage and UFW. The role of the meter reader is therefore important and should encompass the following:

- Checking of the lead seals (Where they exist)
- Ensuring that the meter is still working satisfactorily
- Checking for visible leaks on the PHE side as well as the customer's side of the meter
- Checking for fraudulent activity
- Checking that the consumption is what was expected

(b) Meter Installation/Replacement

There is little doubt that a substantial volume of water and hence income is being lost due to defective meters. All defective meters commencing with those that would generate the most income should be replaced immediately. Meters should be tamperproof and installed, calibrated, maintained and replaced in accordance with PHE standard procedures. These will need to be specified in a PHE metering policy. House connections are also a major source of water lost through leakage and therefore, a standard specification for house connections should be implemented.

(c) Commercial Meter Sizing

It is often the case that Commercial/industrial meters are oversized. This is sometimes due in part to ensure sufficient flows for fire fighting purposes. Large meters do not capture the low flows and therefore there could be significant lost revenue. A review of meter sizes should be made to ensure that low flows are captured and that correctly sized meters are installed accordingly.

5.4.4 NRW Reduction Action Plan

Based on the above and experience gained from the ‘NRW Reduction Pilot Project’, PWD will need to consider implementation of the key activities associated with ensuring NRW reduction as detailed in the ‘NRW reduction Action Plan’. Refer to Volume III Chapter11 Recommendations and Actions Should be Taken by PHE.

CHAPTER 6

MASTER PLAN FOR SANITATION

CHAPTER 6 MASTER PLAN FOR SANITATION

6.1 Basic Condition for Master Plan

6.1.1 Target Year

The target year of this study, Master Plan, is 2025 in accordance with the result of the discussion held on 11th March 2005 between the Study Team and the executing agency, which is set out in "the Minutes of Meeting on the Inception Report".

6.1.2 Selection of Sanitation System

(1) Study Area

The Study Area for sanitation improvement includes:

- Panaji and its surroundings,
  - Taleigao
  - Caranzalem
  - Dona Paula
  - Ribandar
  - St. Cruz
  - Merces
  - Porvorim
- Margao
- Ponda
- Mapusa
- Coastal Belt of South Goa
- Coastal Belt of North Goa

The Study Area is shown in Figure 61.1.

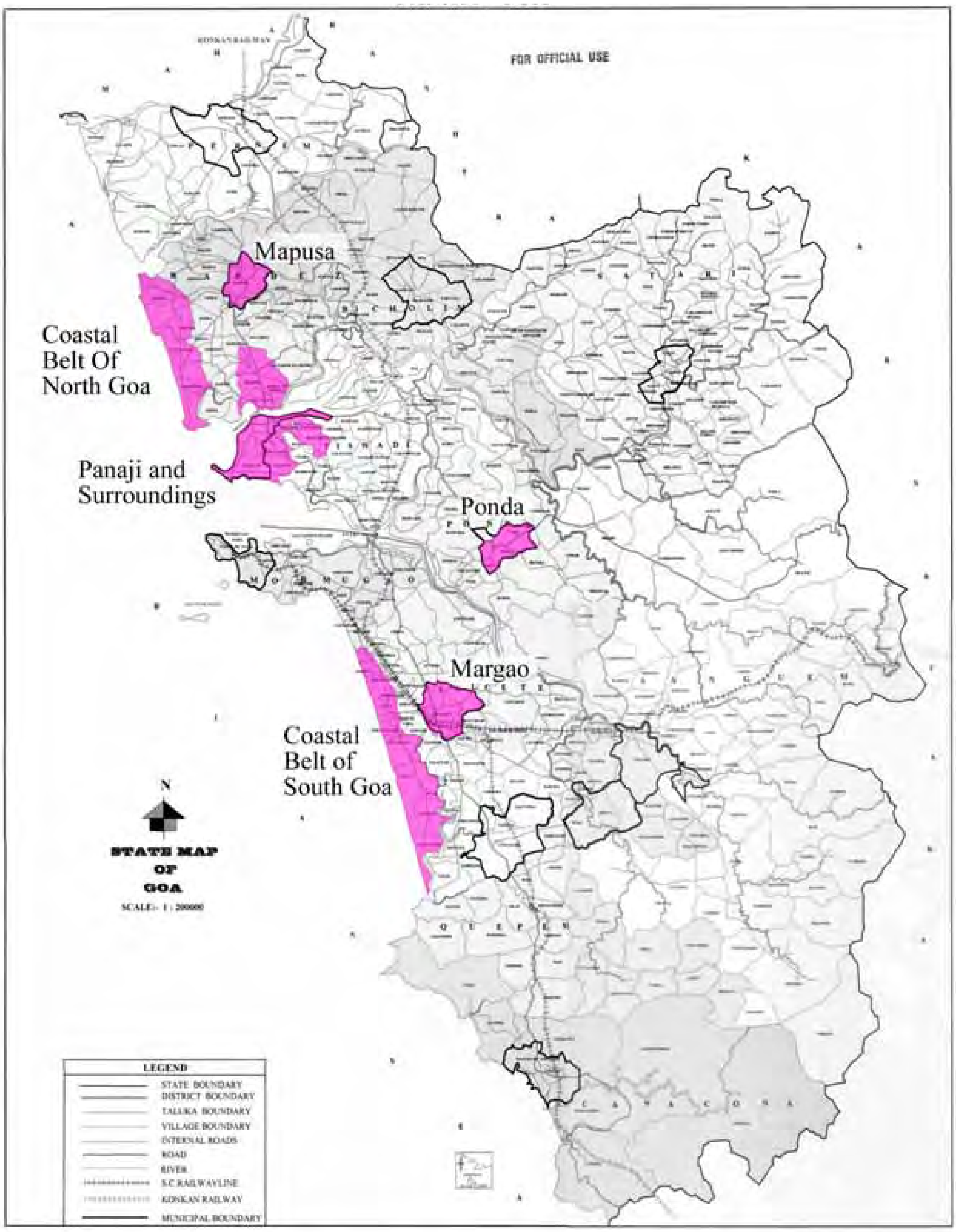


Figure 61.1 Location of Study Area

(2) Selection of Sanitation System

Appropriate sanitation system is selected for each area considering the following demographical, geological and economical factors. Flow chart of the selection procedure is shown in Figure 61.2.

- The treatment system applicable in the study area is conventional sewerage system, decentralized system and onsite (septic tank with soak pit) system.
  - Onsite system: a natural system or mechanical device used to collect, treat, and discharge or reclaim sewage from an individual dwelling without the use of community-wide sewers or a centralized treatment facility. A conventional onsite system includes a septic tank with a soak pit.
  - Decentralized system: A decentralized system is a wastewater collection and treatment system that serves two or more dwellings. Individual septic tanks or aerobic units may pretreat wastewater from several homes before it is transported through low cost, alternative sewers to a treatment unit that is relatively small compared to centralized systems.
  - Sewerage system: A sewerage system is a centralized sewage collection and treatment system using conventional sewers to collect and transport sewage to a relatively large treatment plant.
- The groundwater table in the Study area is more than one (1) meter deep, thus soak pits could be built and operated effectively (Volume IV Appendix M61.1 Study on Infiltration from Soak Pit in Relation with Groundwater Level). Some previous studies observed poor infiltration of soil and many soak pits in trouble, and indicated that the soil condition in a part of the Study area was not suitable for soak pits.
- In densely populated area, it is difficult to secure sufficient area to build large enough soak pits to receive generated sewage flow. From analysis of long-term infiltration rate in relation to population density and sewage flow, the Study Team proposed a practical threshold population density that construction of onsite system becomes not feasible. (Volume IV Appendix M61.2 Long Term Infiltration for Soak Pit)
- A threshold population density that sewerage system becomes economical than onsite system has been proposed based on construction cost comparison between onsite system and sewerage system. (Volume IV Appendix M61.3 Study on Cost Threshold Sewer Length)
- For small cities with population density over the above threshold, but total sewage flow less than 1,000 m^3/day, a decentralized system is proposed as the most

economical option for the residents.

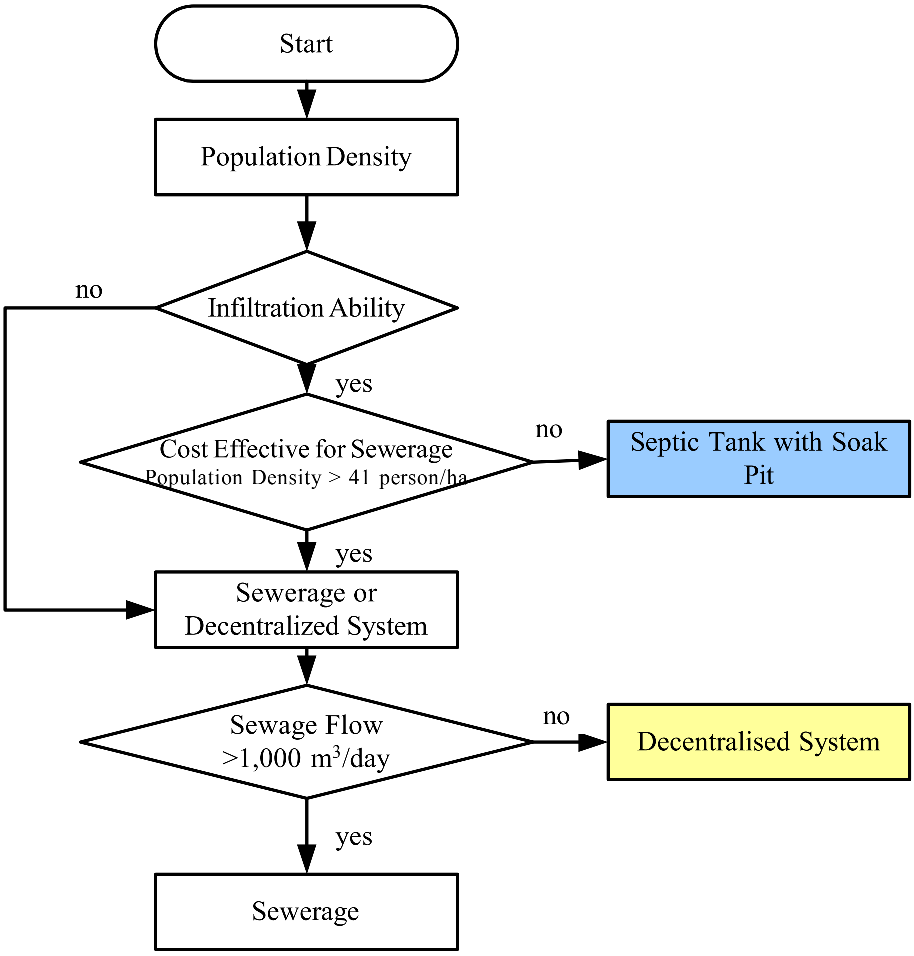


Figure 61.2 Selection Flow Chart of Sanitation Systems

Through the criteria foresaid, the recommended area to be covered by sewerage and decentralized systems are listed below.

Cities where sewerage system is proposed due to insufficient soil infiltration rate for soak pit (Population density more than 213/ha)

- Mapusa,

Cities where sewerage system is more economical than onsite system (Population density more than 41/ha)

- Taleigao, Dona Paula & Caranzalem, Ribandar, Merces, Porvorim
- Colva, Adsulim (Coastal belt of South Goa)
- Calangute, Candolim (Coastal belt of North Goa)

Small Cities where decentralized system is proposed (Population density more than 41/ha, sewage flow less than 1,000 m^3/day)

- Rebandar, Merces, Adsulim

41 person/ha is figured out based on the comparison study on construction cost of the onsite and sewerage facilities.

The proposed treatment system for each area is shown in Figures 61.3 and Table 61.1.

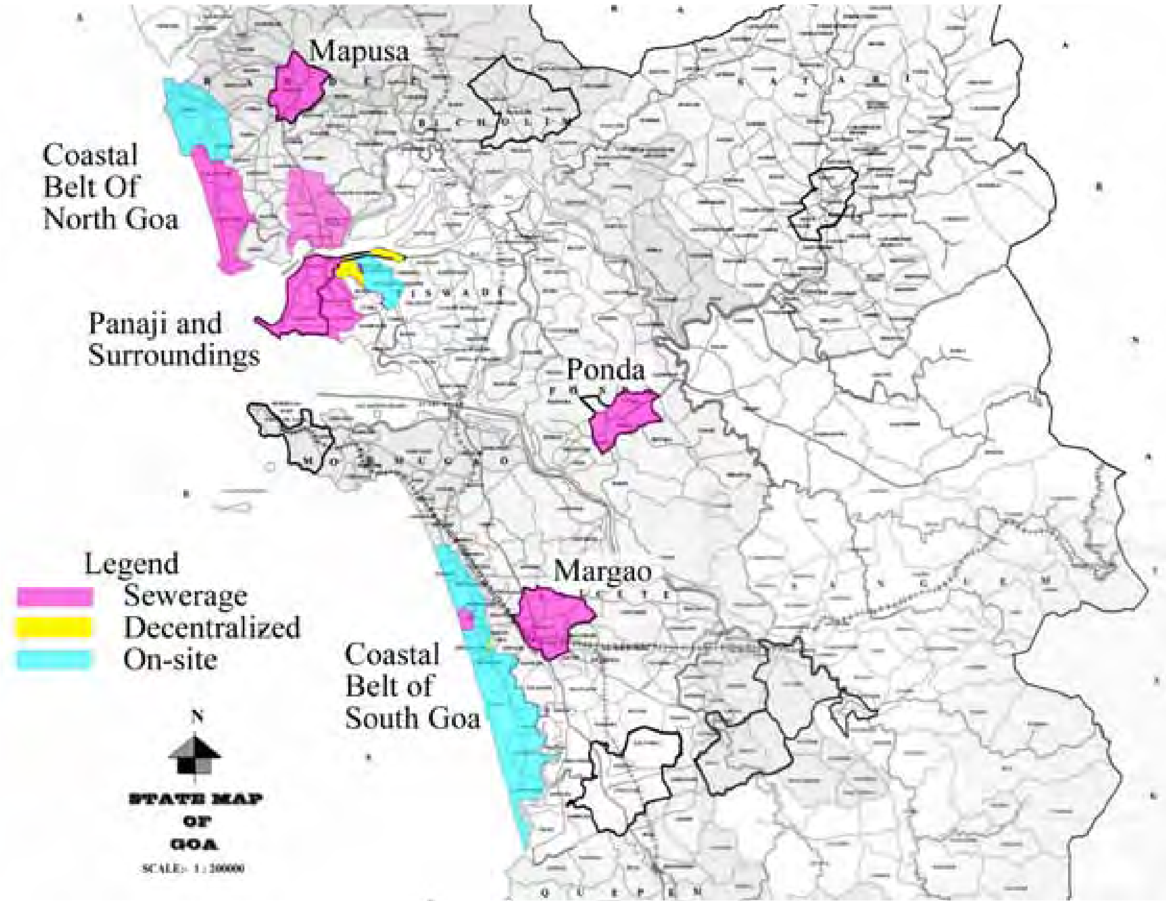


Figure 61.3 Selected Sanitation System in the Study Area

Table 61.1 Selection of Sanitation System

Area			Sanitation System		
			Sewerage	Decentralized	Onsite
Panaji	City		○		
	Others	Taleigao	○		
		Dona Paula & Caranzalem	○		
		Ribandar		○	
	St. Cruz	Calapor	○		
	Merces	Moranbi O Grandi			○
		Moranbi O Pequeno		○	
	Porvorim		○		
Margao			○		
Ponda			○		
Mapusa			○		
Coastal Belt of South Goa		Colva	○		
		Adsulim		○	
		Cavelossim, Carmona, Orlim,Varca, Benaulim, Cana, Sernabatim, Vanelim, Gandaulim, Betalbatim, Gonsua, Majorda, Utorda			○
Coastal Belt of North Goa		Candolim, Calangute	○		
		Anjuna, Aprora			○

(3) Sewerage Service Area Selection

Sewerage service area is selected within the selected areas where sewerage system is appropriate, based on town planning and demographic information.

- Residential area is identified based on the village maps, later confirmed by the field survey.
- For the remote communities located distant from the city centre, sewerage or decentralized system is selected only when it is more economical than onsite system. The rest of the area will be served by onsite system.

1) Panaji and Surroundings

Sewerage system was proposed for five (5) areas, namely Taleigao, Dona Paula & Caranzalem, St. Cruz and Porvorim. Dona Paula, Taleigao and Caranzalem belong to Panaji municipality now and is geographically close to the existing Panaji STP, thus collected sewage will be transported to the Panaji STP.

St. Cruz and Porvorim are located far from the Panaji STP and separated by a large river, thus a centralized treatment plant is proposed for each area.

a) Dona Paula, Taleigao, Caranzalem

These areas have been developing rapidly in recent years as bedroom community for Panaji, sewerage service area was selected based on the current and future land use.

b) St. Cruz

The existing residential area is to be covered by sewerage system, but communities distant from the town centre are excluded from the sewerage area.

c) Porvorim

Porvorim consists of three towns and three villages, where population grows rapidly because of their proximity to Panaji. Topography, demographic data and future town plans are taken into account for selection of sewerage service area. The proposed sewerage system mainly collects sewage from plateau area, however, part of low-lying area adjacent to the proposed STP may be covered.

2) Margao

The ground level of the catchment in Margao falls gradually in a generally southern direction. Only two major pump stations are required to transport all the sewage to the STP in the south zone, whole area is selected to be covered

3) Ponda

There are many uphill and downhill in Ponda, many pumping stations are necessary to collect sewage otherwise sewer should be laid in the river bed. Observing the river conditions, there is no space to construct sewer. As a result, more than ten (10) small pump stations are required, whole Ponda is covered by sewerage due to its high population density

4) Mapusa

In Mapusa city, the catchment has a gentle slope from city centre to the proposed STP site. The city centre is surrounded by undulating hilly areas. Some small communities in this area are not covered by the sewerage system.

5) Coastal Belt of South Goa

Colva is the most popular beach resort in the south Goa coast receiving large seasonal local and foreign tourist population. There are many small and medium hotels found along the coast. The proposed sewerage system will cover the existing city area of Colva.

6) Coastal Belt of North Goa

Calangute & Candolim represents beach resort area in the north Goa coast. It also has many small and medium-scaled hotels. The proposed sewerage will cover the existing built-up area.

6.1.3 Sewage Collection System

Separate system should be applied in the target areas of this project that is only sanitary sewers should be installed, because most part of the target area is covered with existing drainage system with adequate capacity and few additional sewers would be needed. Further, the provision of combined sewers requires longer construction period as well as greater cost than separate system. From the viewpoint of the water quality in the public water body, separate system is more adequate because some part of combined sewage may be discharged into public water body during the rainy season. The diameter of combined sewer is larger and the flow velocity is smaller than separate system, the sluggish flow in the dry season leads deposition and it causes foul odors.

6.1.4 Design Population

(1) Residential Population

Residential population for each area is shown in Table 61.2, which is calculated as follows.

- Population of the service area estimated from village maps and field survey
- Population of the service blocks estimated using population densities of municipalities, towns and village. Population densities of wards were also used whenever available.

Table 61.2 Residential Population

Unit: Person

Study Area		2015	2025
Panaji		30,080	30,413
Panaji Surroundings	Taleigao, Dona Paula & Caranzalem	25,858	26,144
	St. Cruz	14,260	16,918
	Porvorim	37,749	47,848
Margao		99,602	118,193
Ponda		18,678	19,401
Mapusa		54,329	68,255
Coastal Belt of South Goa	Colva	4,590	5,279
Coastal Belt of North Goa	Calangute & Candolim	31,838	39,358
Total		**316,984**	**371,809**

(2) Tourist Population

The peak tourist population observed in December is used to calculated design sewage flow. O&M costs are calculated based on yearly average tourist population.

- Cumulative tourist population is calculated assuming domestic tourist stays for 5 days per visit while foreign tourist stays for 9 days.
- The projected future tourist population for the Study Area is calculated based on the tourist projection of the whole Goa state, which is distributed to each Talka according to the tourism census. Tourist population is further divided into that of towns and villages according to number of the hotel beds.
- The projection of the future tourist population for the Goa State in set out in Volume II Chapter 4 Section 1.
- Data related with tourists, i.e. village wise tourist, monthly fluctuation, projected future tourist, are shown in Volume IV Appendix M61.5 Tourists Population.

(3) Design Population

Residential and tourist population in the proposed sewerage service area is summarized in Table 61.3.

Table 61.3 Design Population for Sewerage Service

		Population (Person)	Tourist (Person/day)
		2025	2025
Panaji		30,413	24,839
Panaji Surroundings	Taleigao, Dona Paula & Caranzalem	26,144	8,737
	St. Cruz	16,918	-
	Porvorim	47,848	1,653
Margao		118,193	5,429
Ponda		19,401	2,097
Mapusa		68,255	1,703
Coastal Belt of South Goa	Colva	5,279	5,231
Coastal Belt of North Goa	Calangute & Candolim	39,358	20,261
Total		**371,809**	**69,950**

6.1.5 Sewage Unit Flow

(1) Domestic and Institutional Sewage Unit Flow

The unit sewage flow from domestic and institutional origin is calculated based on the following assumptions.

- The sewerage service area mostly covers urban areas. Although some service areas are classified as rural areas, these areas are already urbanized and expected to officially become urban area in the near future. Thus unit sewage flow for the urban area (150 ℓ/day/capita) is used for the whole sewerage area.
- Sewage of commercial origin is regarded mainly from tourism industry, thus calculated as tourism sewage flow.
- Drinking water use by the governmental and municipal institutions is 0.34%～5.05% (Average 2.1 %) of domestic water usage. Three (3) percent of domestic sewage flow is used as institutional sewage flow allowing fluctuation in usage. (Refer to Volume IV Appendix M61.6 Sewage Unit Flow)
- Unit sewage flow is calculated at 80 percent of water supply flow following CPHEEO, except for Panaji area, in which the sewage flow to the existing STP is 100 percent of the water supply flow. (Refer to Volume IV Appendix M61.7 Study of Panaji STP Sewage Inflow)
- Infiltration to sewer is calculated at 20 percent of domestic sewage flow as specified by CPHEEO. (Refer to Volume IV Appendix M61.8 Groundwater Infiltration)

The calculated domestic and institutional unit sewage flow is shown in Table 61.4.

Table 61.4 Per Capita Sewage Flow(Domestic and Institutional Sewage Flow including Groundwater Infiltration)

Unit: ℓ/day/capita

	2010-2025
Domestic Water Use	150
Ratio of Institutional Water Use	3%
Institutional Water Use	4.5
Total	154.5
Sewage Return Ratio	80%
Sewage Flow	123.6
Groundwater Infiltration Rate	20%
Groundwater Infiltration	25
Sewage & Groundwater	148
Say	150

(2) Tourism Sewage Unit Flow

Tourism water demand is estimated based on CHEEEO and 80 percent of water flows into sewerage system. Table 61.5 shows the per capita tourism sewage flow.

Table 61.5 Per Capita Tourism Sewage Flow

Hotel Rank	Number of Beds	Water Use (ℓ/day/tourist)
Above 5 Star	4,413	450
3 & 4 Star	2,963	340
1 &2 Star	4,065	180
Total/Average	**11,441**	**326**
Sewage Return Ratio		**80%**
Average Sewage Flow		**260**

(3) Industrial Wastewater and Defense Sewage

Industrial wastewater and defense sewage are estimated based on the water demand. Industrial wastewater flow calculated from water consumption data in Margao is shown in Table 61.6. A large part of breweries & bottling factory's water consumption is consumed for their products. The remains are assumed to be used for domestic consumption by workers, and 80 percent of them become sewage. Overall, 35 percent of industrial water demand is estimated to become sewage. For defense sewage, 80 percent of defense water demand is estimated to become sewage.

Table 61.6 Estimation of Industrial Wastewater Return Ratio

Unit: m^3/day

Factory No.	Water Meter				Discharge	Sewerage	Remarks
	1	2	3	Total		80%	
1	2	1	1	4	4	3.2	Others
2	12	5		17	17	13.6	Others
3	38	199		237	38	30.4	Bottling
4	19	109		128	19	15.2	Breweries
5	90	23	55	168	168	134.4	Others
Total				554	246	196.8	
Sewage per water consumption						35.5% ≒35%	

Source: PWD Margao

6.1.6 Design Sewage Quantity

(1) General

The proposed sewage treatment plants are designed for the average domestic and institutional sewage flow and the peak tourism sewage flow observed in December, following CHEEEO. Sewer and pump stations are designed for hourly peak flow, which is calculated by multiplying the design sewage flow for STP (F_{stp}) by the peak factors shown in Table 61.7.

Operation costs are calculated based on the average domestic, institutional and tourism sewage flow (F_{ave}).

Table 61.7 Peak Factor

Population of city	Peak Factor
Up to 20,000	3.0
20,000 to 50,000	2.5
50,000 to 750,000	2.25
Above 750,000	2.0

Source: CPHEEO

(2) Domestic and Institutional Sewage Flow

Table 61.8 shows domestic sewage flow for each service area based on per capita domestic and institutional sewage flow.

Table 61.8 Domestic and Institutional Sewage Flow

Unit: m^3/day

Study Area	2015	2025
Panaji	9,443	9,547
St. Cruz	2,139	2,538
Porvorim	5,662	7,178
Margao	14,940	17,729
Ponda	2,802	2,910
Mapusa	8,149	10,238
Colva	689	792
Calangute & Candolim	4,776	5,904
Total	48,600	56,836

*Including groundwater infiltration

(3) Tourism Sewage Flow

Tourism sewage flow is calculated based on the unit sewage flow and the projected tourist population for each service area. The results are shown in Table 61.9.

Table 61.9 Tourism Sewage Flow

Unit: m^3/day

Study Area	Average		Peak Season	
	2015	2025	2015	2025
Panaji	3,839	5,605	7,102	10,370
St. Cruz	0	0	0	0
Porvorim	159	232	294	430
Margao	523	763	967	1,412
Ponda	202	295	373	545
Mapusa	164	239	303	443
Colva	504	735	932	1,360
Calangute & Candolim	1,950	2,848	3,608	5,268
Total	7,341	10,717	13,579	19,828

(4) Other Sewage Flow

Industrial wastewater and defense sewage are calculated based on the return ratio described in Section 6.1.5, and summarized in Table 61.10.

Table 61.10 Industrial Wastewater and Defense Sewage Flow

Unit: m^3/day

Category	Town	Water Demand		Sewage Flow	
		2015	2025	2015	2025
Industrial	Margao	1,015	2,656	355	930
	Mapusa	108	286	38	100
Defense	Panaji	1,166	1,473	1,166	1,473
	Margao	792	985	634	788
Total		3,081	5,400	2,193	3,291

(5) Summary of Design Sewage Flow

The design sewage flow is summarized and shown in Table 61.11 and the detail is shown in Volume IV Appendix M61.9 Sewage Flow.

Table 61.11 Summary of Design Sewage Flow at 2025

Unit: m^3/day

Study Area	Domestic & Institution	Tourism	Others	Total
Panaji	9,547	10,370	1,473	21,390
St. Cruz	2,538	0	0	2,538
Porvorim	7,178	430	0	7,608
Margao	17,729	1,412	1,718	20,859
Ponda	2,910	545	0	3,455
Mapusa	10,238	443	100	10,781
Colva	792	1,360	0	2,152
Calangute & Candolim	5,904	5,268	0	11,172
Total	56,836	19,828	3,291	79,955

6.1.7 Design Sewage Quality

(1) Domestic Sewage Pollution Load

The per capita sewage pollution loads are BOD 45 g/day/capita and SS 90 g/day/capita in the CPHEEO. Compare with Japanese guideline, the BOD value is smaller and the SS value is much larger. The per capita BOD load in CPHEEO is reasonable because the water consumption in India is smaller (150 ℓ/day) than that in Japan (200 - 250 ℓ/day), the pollution load derived from gray water may be decreased. On the contrary, SS load or SS/BOD is different from not only Japanese guideline but also from water examination record in Panaji.

The sewage in the study area is assumed similar to that of Panaji. The SS/BOD ratio of 85 percent is adopted based on the maximum ratio recorded by the past sewage quality analysis at Panaji STP, shown in Table 61.13. Applied per capita BOD and SS are shown in Table 61.12.

Table 61.12 Per Capita Domestic Sewage Pollution Load

Unit: g/day/capita

	Applied	CPHEEO	Japanese Guideline			Remarks
Item	Total	Total	Total	Breakdown		
				Black water	Gray water	
BOD	45	45	58	18	40	
SS	38	90	45	20	25	
SS/BOD	85%	200%	78%			

Table 61.13 Raw Sewage Quality Examination Record at Panaji STP

Unit: mg/ℓ

Parameter	May-Jun, 2002	Jan, 1999	Jan-Dec, 1998	Average	Remarks
BOD	183	208	189	193	
SS	121	176	113	137	
SS/BOD	66%	85%	60%	71%	

(2) Tourism Sewage Pollution Load

There is no description about the pollution load of tourism sewage in CPHEEO. The per capita tourism sewage pollution load is assumed as the same as the domestic sewage load because the average length of stay in the target area is rather long, which ranges from 5 days (domestic) to 9 days (foreign). Per capita tourism BOD and SS is shown in Table 61.14.

Table 61.14 Per Capita Tourism Sewage Pollution Load

Unit: g/day/capita

Parameter	Tourism Sewage	Remarks
BOD	45	
SS	38	

(3) Institutional, Industrial and Defense Sewage Pollution Load

Institutional and defense sewage are mainly composed of labor sewage and assumed as same quality as domestic sewage and shown in Table 61.15.

Table 61.15 Institutional, Industrial and Defense Sewage Quality

Unit: mg/ℓ

Parameter	Institutional, Industry and Defense	Remarks
BOD	300	45(g/day/capita) / 150 (ℓ/day/capita) x 1000
SS	255	38 / 150 x 1000

(4) Total Pollution Load and Sewage Quality

The area wise sewage flow and pollution load are shown in Table 61.16. Sewage quality is also shown in the same table obtained by dividing pollution load by sewage flow. The breakdown is shown in Volume IV Appendix M61.10 Pollution load and Sewage Quality.

Table 61.16 Total Pollution Load and Sewage Quality

Area	Quantity	Pollution Load (kg/day)		Quality (mg/ℓ)	
	(m^3/day)	BOD	SS	BOD	SS
Panaji	21,390	4,499	3,802	210	178≒180
St. Cruz	2,538	761	643	300	253≒250
Porvorim	7,608	2,228	1,881	293≒300	247≒250
Margao	20,859	6,078	5,135	291≒300	246≒250
Ponda	3,455	967	817	280	236≒240
Mapusa	10,781	3,178	2,685	300	249≒250
Colva	2,152	473	400	220	190
Calangute & Candolim	11,172	2,683	2,266	240	200

(5) Design Treated Effluent Quality

The treated effluent shall comply the Indian standards for discharge of sewage. Table 61.17 presents the standard BOD and SS values of effluent.

Table 61.17 Design Effluent Quality

Unit: mg/ℓ

Parameter	Design Effluent Quality	Remarks
BOD	Not Greater Than 30	Standards for Discharge of Sewage
SS	Not Greater Than 100	Standards for Discharge of Sewage

Source: Parivesh - Sewage Pollution- Central Pollution Control Board, Ministry of Environment & Forests, February 2005

6.2 Design Criteria for Sewerage Facilities

6.2.1 Sewer Network

Design criteria used for the preliminary design of trunk sewers are mainly in accordance with the "Manual on Sewerage and Sewage Treatment (Central Public Health and Environmental Engineering Organization, Ministry of Urban Development of India, December 1993)". Criteria adopted for design of sewer are shown in Tale 62.1. and the comparison between Indian criteria and Japanese criteria is shown in Volume IV Appendix M62.1 Design Criteria for Sewer Network.

Table 62.1 Design Criteria for Sewer

Item	Criteria
(1) Design flow	Peak flow
(2) Flow formula	
Gravity flow	Manning formula (Concrete pipe: n=0.015)
Pressure flow	Hazen-Williams formula (Cast iron: C=100)
(3) Depth of flow	0.8 of full at ultimate peak flow
(4) Minimum Velocity	0.8 m/sec
(5) Maximum Velocity	3.0 m/sec
(6) Minimum diameter of sewer	150 mm
(7) Minimum depth of earth covering	1.0 m for branch sewer 1.5 m for gravity trunk sewer
(8) Pipe materials	Concrete pipe for gravity sewer Cast iron pipe for pressure main
(9) Manhole spacing	Diameter of pipe — Manhole spacing Up to 900 mm — < 30 m 900 - 1,500 mm — 90 - 150 m 1,500 - 2,000 mm — 150 - 200 m
(10) Manhole size	For depths above 0.90m and up to 1.65m =900 mm dia. For depths above 1.65m and up to 2.30m =1,200 mm dia. For depths above 2.30m and up to 9.00m =1,500 mm dia. For depths above 9.00m and up to 14.00m =1,800 mm dia. and・Width/diameter of the manhole should not be less than internal diameter of (the sewer + 150mm on both sides)

6.2.2 Pumping Station

Design criteria for design pumping station are also mainly in accordance with the “Manual on Sewerage and Sewage Treatment”. Criteria adopted for design of pumping station are shown in Tale 62.2 and the comparison between Indian criteria and Japanese criteria is shown in Volume IV Appendix M62.2 Design Criteria for Pumping Station.

Table 62.2 Design Criteria for Pumping Station

Item	Criteria		
(1) Design flow	Peak flow. Standby pump capacity is 50% of peak flow		
(2) Type of pumping station	Manhole type (design flow ≦ 3.0 m^3/min)		
	Conventional type (design flow > 3.0 m^3/min)		
(3) Screen facility	Bar spacing less than 20 mm		
(4) Type of pump equipment	Submersible type		
(5) Composition of pump equipment			
Manhole type	2 unit (including 1 standby)		
Conventional type	3 - 6 m^3/min	3units	1/2Q×3units (1)
	6 - 12 m^3/min	4units	(1/4Q×2units) + ('2/4Q×2units(1))
	12 - 24 m^3/min	5units	(1/8Q×2units) + ('2/8Q×1unit) + ('4/8Q×2(1)units)
(6) Specification of pump equipment	(1) Pump diameter D = 146×(Q/V)$^{0.5}$ Where D : Pump inlet/outlet diameter Q : Flow-rate (m^3/min) V : Velocity (=1.5 - 3.0 m/sec) (2) Motor power of pumps P = (0.163×Q×H/n)×(1+α) Where P : Motor power (kw) Q : Discharging flow (m^3/min) H : Pump head (m) n : Pump efficiency (60 - 85%) α : Allowance of motor power (= 0.15)		
(7) Minimum size of pump	100 mm in diameter		

The pumping stations of conventional and manhole type are shown in Figures 62.1 and 62.2.

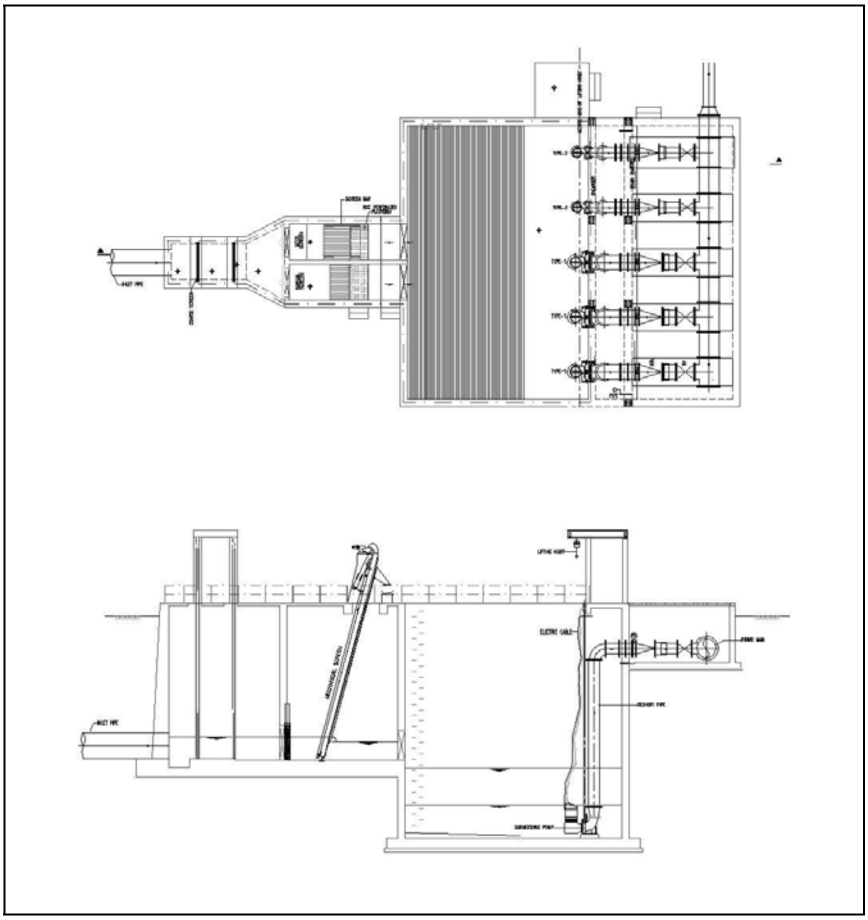


Figure 62.1 Typical Conventional Type Pumping Station

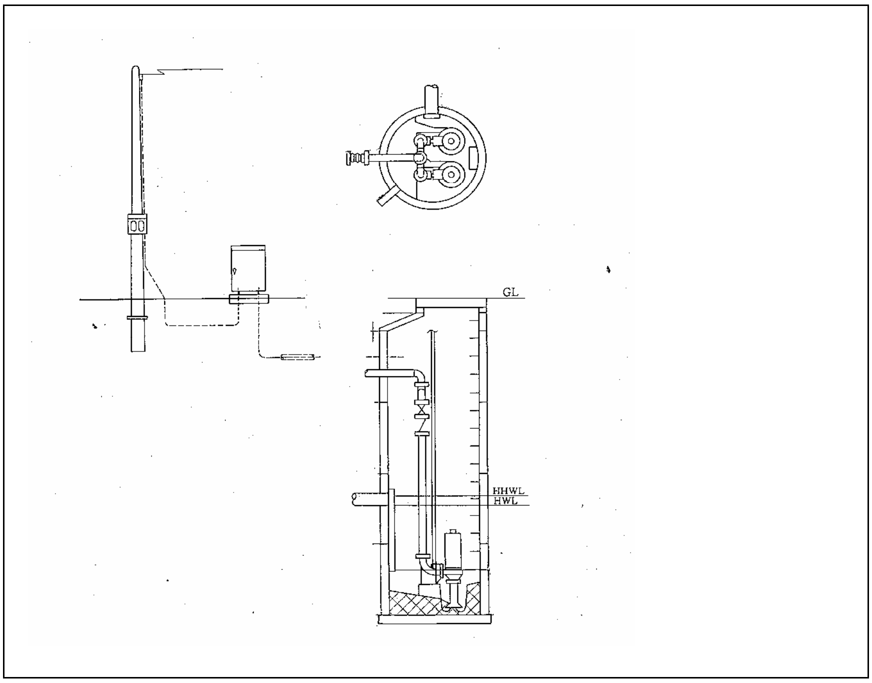


Figure 62.2 Typical Manhole Type Pumping Station

6.2.3 Sewage Treatment Plant

(1) Selection of the Location

New STP sites shall be selected for sewerage service blocks except Panaji (Tonca) and Margao blocks, which have the existing STPs. The following selection criteria will be used for new STP sites.

- Topography
- Ease in land acquisition
- Environmental and social considerations in adjacent areas
- Protection of STP facilities from flooding

(2) Requirements for Sewage Treatment Plant

a) Design Sewage Flow and Sewage Quality

The proposed STPs are designed for projected sewage flow and quality in 2025, which is discussed in Chapter 6.1 and summarized in Table 62.3.

Table 62.3 Design Flow and Quality

Sewage Treatment Plant	Design Flow (m^3/day)	Design Quality (mg/l)	
		Inlet Raw Sewage	
		BOD	SS
Panaji (Tonca)	21,400	210	180
St. Cruz	2,500	300	250
Porvorim	7,600	300	250
Margao	20,900	300	250
Ponda	3,500	280	240
Mapusa	10,800	300	250
Colva (South Coastal Belt)	2,200	220	190
Baga (North Coastal Belt)	11,200	240	200
Total	80,100	-	-

Note: Design flow shows as daily average flow

b) Effluent Water Quality Standards

Sewage treatment method should be selected so that its effluent meets at least the effluent quality standards in India. In case of discharging the treated water to inland surface water, effluent quality standards for major parameters, are presented in Table 62.4.

Table 62.4 Effluent Quality Standards for STP

Parameter	Standard Value
pH	5.5 – 9.0
BOD	max. 30 mg/l
COD	max. 250 mg/l
SS	max. 100 mg/l

(3) Selection of Treatment Methods and Proposed Design Criteria

1) Required Sewage Treatment Level

Sewage treatment systems are categorized into 4 levels, namely preliminary, primary, secondary and advanced treatment, based on treatment efficiency. Appropriate treatment level shall be selected considering applicable effluent discharge standards, current and future use of receiving waters, O&M capacity and available technical expertise in the executing agencies. The advanced treatment is only considered in case that receiving water is used as drinking water source or is a closed water body such as a lake, or that effluent is to be re-used. **The secondary treatment is necessary to discharge treated effluent to the rivers in this project**, although a part of effluent could be re-used. Table 62.5 shows that typical treatment efficiency of primary and secondary treatment.

Table 62.5 Expected Efficiencies of Various Treatment Process

Process	Removal Rate (%)		
	BOD	SS	Total Coliform
Primary treatment	30 - 45	45 - 60	40 - 60
Secondary treatment			
High rate trickling filters	75 - 80	75 - 85	80 – 90
Activated sludge	85 - 95	85 - 90	90 – 96
Stabilization ponds	90 - 95	80 - 90	90 - 95

Source: CPHEEO manual p.199

2) Proposed Sewage Treatment Methods for Panaji and Margao

The proposed extension of the existing sewage treatment plants in Panaji and Margao will used the same treatment process as used for the existing plants with the following reasons.

- Secondary treatment is adopted for these existing plants.
- Two different treatment systems within the treatment plant may confuse operators, thus induce them to make mistakes in operation and maintenance.

Thus, the treatment methods are decided as follows.

Panaji (Tonca) = SBR method

Margao = Conventional Activated Sludge method

3) Applicable Sewage Treatment Processes for New Treatment Plant

New sewage treatment plants are proposed for St. Cruz, Porvorim, Ponda, Mapusa, South and North Coastal Belt. Appropriate treatment processes will be selected based on the following conditions.

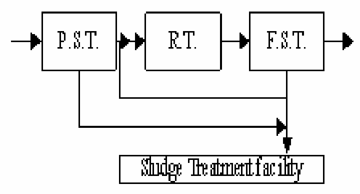
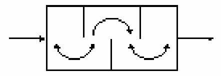
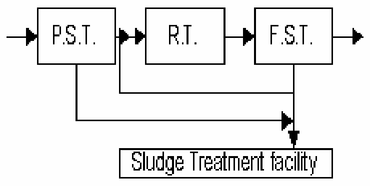
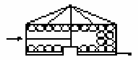
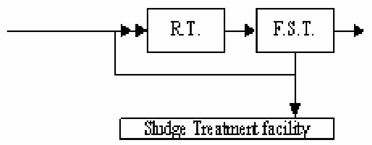
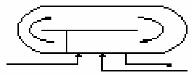
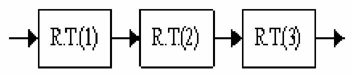
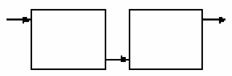
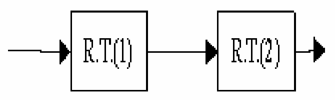
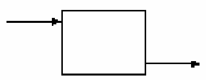
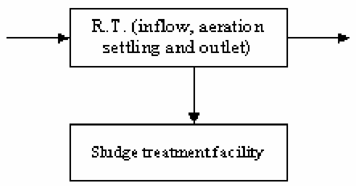
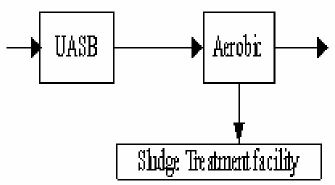
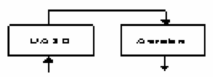
- Experience in India and Goa State
- Easy O&M
- Effect on the surrounding areas (odor, noise, landscape, flies, etc.)
- Least cost for construction and O&M
- Required land area

Major secondary treatment processes and their applicability are shown in Table 62.6, and applicable processes are compared in Table 62.7.

Table 62.6 Major Secondary Treatment Processes

Type	Common Name of Treatment Method	Applicability
1. Aerobic processes		
(1) Suspended growth	Activated sludge processes	
	- Conventional activated sludge (CAS)	△
	- Oxidation ditch (OD)	◎
	- Sequencing batch reactor (SBR)	◎
	- Complete mixed	-
	- Pure oxygen	-
	- Extended aeration (EA)	-
	Aerated lagoons (AL)	◎
(2) Attached growth	Trickling filters	○
	Rotating biological contactors	-
2. Anaerobic processes		
(1) Suspended growth	Upflow anaerobic sludge blanket (UASB)	◎
	Anaerobic digestion	-
	Anaerobic contact	-
(2) Attached growth	Anaerobic filters	-
3.Pond processes	Stabilization Ponds (SP)	△
	- Anaerobic ponds	
	- Facultative ponds	
	- Maturation ponds	

Table 62.7 General Comparisons for Applicable Sewage Treatment Method

Treatment Method	Composition of the Sewage Treatment Process	Theory of Reaction in Reactor Tank	Features of Sewage Treatment Process	Features of Treatment Efficiency	Operation and maintenance	Applicability
1. Conventional Activated Sludge (CAS method)		 Sewage flows down together with activated sludge organic substance is absorbed and assimilated by activated sludge.	Retention time in reactor tank is relatively short and load is high. Thus, primary sedimentation tank is needed to cope with the fluctuation in sewage flow and quality and to equalize/ mitigate the load. Sludge treatment facility is necessary as well.	* BOD removal rate is 85-95% * Transparency of treated water is high * Stability in sewage temperature fluctuation is inferior in comparison with other methods * Generated sludge volume is larger than other methods	* The system has many maintenance and inspection points. Thus, advanced/complicated operational technique is needed	△
2. Trickling Filter (TF method)		Section  Sewage is sprinkled on bio-filter by rotating distributor. Contained organic substance is absorbed/ assimilated by bacteria attaching on bio-filter. Enlarged bacteria membrane falls out and are	Primary sedimentation tank must be installed to prevent clogging in bio-filter and distributor's nozzle. Sludge treatment facility is needed as well.	* BOD removal rate is 75-90% * Transparency of treated water is worse than CAS * Less affected by sewage temperature fluctuation compared with CAS * Flies and odours are generated	* O&M is easy since no advanced /complicated operational technique is needed * Attention must be paid to fly/odour generation	○
3. Oxidation Ditch (OD method)		 Sewage is circulated together with activated sludge and contained organic substance is absorbed and assimilated by activated sludge.	This process is flexible to the fluctuation of sewage flow and quality by its long retention time in reactor tank. Primary sedimentation tank is not necessary, however, sludge treatment facility is needed.	* BOD removal rate is 75-95% * Denitrification is possible by operational condition	* Operational technique is easier than the CAS method and same level as EA method	◎
4. Stabilization Ponds (SP method)		 Sewage is purified by oxidation of aerobic bacteria activated by oxygen supply through algae or anaerobic bacteria.	Since oxygen supply in reactor tank is conducted by natural oxidation and photosynthesis of algae. Retention time is extremely long. Sludge treatment facility is not needed. Anaerobic pond, maturation pond and aerobic pond are allocated individually or combined.	* Although BOD removal rate is affected by sewage temperature and retention time, approximately 70-90% removeal is possible * Stability in sewage flow and temperature fluctuation is relatively good but once deteriorated, recovery takes a long time * Odours and harmful insects are generated	* Easiest process in O&M * Algae control is important for stable efficiency * Ponds is drained periodically, once in 1-5 years. Sludge is carried out to site/disposed of after drying by sun light	△
5. Aerated Lagoons (AL method)		 Sewage is purified by oxidation of aerobic bacteria.	Since oxygen supply in reactor tank will be done by compulsive oxidation, retention time is shorter than that of flowing Stabilization Pond process. Sludge treatment facility is not needed.	* BOD removal rate is affected by sewage temperature and retention time as well as the SP method, the rate will be 70-90% * Stability in load fluctuation is superior * Less odour generate than SP method	* O&M is easy since there is simple equipment	◎
6. Sequencing Batch reactor (SBR method)		 The processes of (1)Inflow, (2)Aeration, (3)Sedimentation, (4)Effluent and sludge drawing are occurred in the one tank	SBR process is a fill-and-draw activated sludge treatment system. Aeration and sedimentation are carried out in the unit tank. SBR is use for small communities where land area is limited. Process is can remove nitrogen and phosphorus.	* BOD removal rate is 75-95% * Transparency of treated water is high * Stability in sewage temperature fluctuation is good * Nitrification is expected	* Operational technique is easier than the CAS method, but difficult compared with OD and EA methods	◎
7. UASB + Aerobic		 UASB: Upflow Anaerobic Sludge Blanket process	-UASB: Sewage treatment occurs through a sludge blanket composed of biologically formed granules.	* BOD removal rate of UASB is approximately 60 %, composite system may be 80 - 90 %. * Generated sludge volume is small * Experience as sewage treatment method is few in the world *	* It is important to keep the granulus in the reactor tank * Operation technique is easier than the CAS method and same as OD and EA method	◎

Legend P.S.T. : Primary Sedimentation Tank R.T. : Reactor Tank F.S.T. : Final Sedimentation Tank

4) Detailed Comparisons of Four Treatment Methods

Four treatment methods, oxidation ditch (OD), aerated lagoon (AL), sequencing batch reactor (SBR) and UASB＋trickling filter (TF) are compared for treating 10,000 m^3/day sewage flow with the above design sewage quality. The results are summarized in Table 62.8 and the detail comparison study is presented in Volume IV Appendix M62.5 Detailed Comparison of Four Treatment Methods.

In the qualitative analysis, the AL method received the highest points and the (UASB+TF) method received the lowest points. The (UASB+TF) method is composed with three (3) unit process such as UASB(anaerobic), trickling filter(aerobic) and settling tank. An anaerobic process often generates odor and flies. Furthermore, (UASB+TF) needs similar sludge treatment as OD/SBR method, only less sludge generates. In these reason, (UASB+TF) show most disadvantageous in “environment” and “ease of operation and maintenance”, it shall be excluded in this comparison. Comparing with OD and SBR, there is no big difference but SBR have not been employed so much in tropic countries, OD is evaluated as more suitable.

The following shows the results of the quantitative analysis.

1.Capital Cost (excluding the land cost)

AL > SBR > OD > (UASB+TF)

2.Capital Cost (including the land cost)

SBR > OD > (UASB+TF) > AL

3.O&M Cost

(UASB+TF) > AL > OD = SBR

Net present value (NPV) of construction and O&M cost for 20 years of each treatment method is calculated and shown in Table 62.9. Regarding NPV with land acquisition cost, AL is more expensive than OD.

The Study Team recommended, and the Indian side agreed to employ the OD method. Because the AL method requires huge land space comparing with the OD, it must be difficult to procure such huge land space around the areas where sewage treatment plants required, and the OD method has advantages in odor problem comparing to the AL method especially for tourism area in North and South Coastal Belts.

Table 62.8 Summary of Detailed Comparison of Four Treatment Methods

Items		OD method	AL method	SBR method	UASB+TF method	Remarks
Qualitative Analysis						
1.Experience in the tropical area		4.5	4.5	3.5	4.0	
2.Treatment efficiency BOD removal rate (%)		5.0 (90 – 95)	4.0 (70 – 90)	5.0 (90 – 95)	4.5 (80 – 90)	
3.Treatment stability Retention time (hrs)		4.0 (20.1)	4.5 (156.3)	3.5 (18.3)	3.0 (11.8)	*1
4.Ease of operation and maintenance	Sewage	4.0	4.5	3.5	3.0	*2
	Sludge	2.0	4.5	2.5	2.5	*3
5.Environment	Odor	4.0	3.5	4.0	3.0	*4
	Noise	4.0	4.0	4.0	4.5	
	Flies, etc.	4.0	3.5	4.0	3.0	*5
Total Point in Qualitative Analysis		**31.5**	**33.0**	**30.0**	**27.5**	
Quantitative Analysis						
6.Capital cost (thousand Rs)	Civil works	72,000	33,700	64,200	87,000	
	M&E works	48,000	22,500	42,800	58,000	
	Sub-total	120,000	56,200	107,000	145,000	
7.Land cost	Land area (m^2)	12,800	78,400	9,600	6,900	
	Unit cost (Rs/m^2)	2,083				
	Sub-total (10^3 Rs)	26,670	163,330	20,000	14,380	
Total (6.+ 7.) (thousand Rs)		146,670	219,530	127,000	159,380	
8.O&M cost (thousand Rs/year)	Electricity	7,358.4	7,095.6	7,358.4	3,547.8	
	Labor	1,590.0	858.0	1,590.0	2,220.0	
	Chemical (chlorine)	202.6	202.6	202.6	202.6	
	Chemical (polymer)	1,368.8	0	1,368.8	889.7	
	Spare parts	1,547.6	835.1	1,547.6	998.6	
	Others	-	-	-	-	
	Sub-total	12,067.4	8,991.3	12,067.4	7,858.7	

<Description>:

*1 For flexibility to receive fluctuated sewage flow, AL has advantage over OD because of its longer hydraulic retention time. Typical OD has 20 hours hydraulic retention time, which enable it to attenuate fluctuation of sewage flow.

*2 For ease of operation and maintenance of sewage treatment, OD is superior since SBR requires maintenance of movable decanter.

*3 For ease of operation and maintenance of sludge treatment, only AL does not require daily sludge treatment. Among the other three methods, only OD requires sludge recycling.

*4 For negative impact of odour on surrounding environment, AL may produce more odour than OD and SBR since there are small anaerobic zones in lagoons, which is only partially mixed by aerators. Anaerobic condition of UASB produces more odour than AL does.

*5 For negative impact of others on surrounding environment, possibility of fly and mosquito breeding was evaluated.

Table 62.9 Net Present Value of Construction Cost and O&M Cost 20 years

(Thousand Rs)

Treatment Method	Without Land Acquisition Cost		With Land Acquisition Cost	
	Cost	NPV Discount Rate=3.1%	Cost	NPV Discount Rate=3.1%
OD	409,400	327,373	436,060	354,033
AL	258,700	202,673	422,010	365,983
SBR	391,200	311,183	411,200	331,183
UASB+TF	360,200	296,450	374,570	310,820

7) Study of Design Criteria for Sewage Treatment Process

Design criteria for the proposed sewage treatment processes, SBR for Panaji STP, conventional activated sludge method for Margao STP and oxidation ditch for the other STPs are adopted from the following national and international design criteria.

- **National design criteria in India (CPHEEO)**
- **Japanese design criteria and Metcalf&Eddy**

(3) Selection of the Sludge Treatment Method and Proposed Design Criteria

1) Applicable Sludge Treatment Processes

Various unit operations and processes employed for sludge treatment aim at stabilization of organic matter and reduction of sludge volume by removing water. While, reduction and stabilization of organic matter are achieved by digestion, incineration and composting, the treatment methods aimed at removal of water from sludge include thickening, dewatering and drying. The commonly used sludge treatment methods and sludge condition and possible disposal methods are shown in Volume IV Appendix 62.6 Selection of Sludge Treatment Method.

2) Selection of the Sludge Treatment Method

The existing sludge treatment in Panaji STP uses sludge storage, mechanical dewatering and supplemental sludge drying beds. The Margao STP has sludge digesters and sludge drying beds. Sludge treatment method for this project shall satisfy the following requirements of the study area.

- Satisfactory operation records in India and the Goa state
- Enabling maximum use of the existing facility
- Enabling sludge volume reduction (introduction of sludge thickening and dewatering)

- Providing operational ease
- Providing stabilized and safe-to-handle sludge for reuse (introduction of sludge digestion)

The proposed sludge treatment methods for each sewerage service blocks are shown in Table 62.10.

Table 62.10 Proposed Sludge Treatment Methods

Name of STP	Proposed Sludge Treatment Methods
Panaji (Tonca) STP	Thickening + Digestion + Mechanical Dewatering + (Drying Beds)
St. Cruz	Thickening + Digestion + Mechanical Dewatering
Porvorim	Thickening + Digestion + Mechanical Dewatering
Margao	Thickening + Digestion + Mechanical Dewatering + (Drying Beds)
Ponda	Thickening + Digestion + Mechanical Dewatering
Mapusa	Thickening + Digestion + Mechanical Dewatering
Colva (South Coastal Belt)	Thickening + Digestion + Mechanical Dewatering
Baga (North Coastal Belt)	Thickening + Digestion + Mechanical Dewatering

3) Design Criteria for Sludge Treatment

Design criteria for sludge thickening, digestion, dewatering and drying beds will be adopted from the following national and international design criteria.

- **National design criteria in India (CPHEEO)**
- **Japanese design criteria and Metcalf&Eddy (2003)**

Appropriate design criteria selected from the above criteria are shown in Volume IV Appendix M62.7 Study on STP Cost for Comparison of Treatment Methods.

(4) Re-use of Treated Effluent and Sludge

1) Re-use of Treated Effluent

In dry season, spraying drinking water for watering plants is commonly observed in the study area. Reuse of STP effluent for this purpose is proposed in order to save precious drinking water resources. The proposed reuse facilities will meet the following requirement of the study area.

- Additional sand filtration is introduced to treat secondary treatment effluent in order to meet the requirement of watering plants.

- Water tankers will distribute reuse water. Reuse water storage will be installed in STPs to store the same volume of a water tanker.
- Secondary treated effluent, which is not reused, will be disinfected and discharged to the public waters.

On the other hand, the process of sand filters is applied to treat secondary treated effluent of the North Coastal Belt STP to contribute to the improvement of water quality in the beach resorts.

2) Re-use of Sludge

Treated sludge shall be reused as much as possible. Industrial wastewater that could be received at the proposed STPs are mostly from food and beverage industries, thus heavy metal contents of incoming sewage is expected to be low enough to enable sludge reuse. Dewatered sludge could be used as fertilizer for agriculture and vegetation.

Typical sludge from oxidation process is rather stable due to its long sludge retention time, thus further digestion of sludge could be omitted. In this project, sludge digestion is introduced in order to achieve stable and safe sludge especially suitable for reuse purpose.

6.3 Master Plan for Sewerage Facilities

6.3.1 Panaji Surroundings

(1) General Description

This area is located in the south of main part of Panaji City. Wastewater generated in this area should be carried to existing Tonca STP in Panaji City through two new trunk sewers, one is Dona Paula Trunk Sewer covering Dona Paula and Caranzalem areas and the other is Taleigao trunk sewer that should collect sewage from Taleigao area. Each trunk sewer should be provided with one pumping station respectively. Covered area, population and wastewater generation under these two zones are shown in Table 63.1.

Table 63.1 Design Basis of Panaji Surroundings

Items	Taleigao	Dona Paula *	Total
Sewerage Service Area (ha)	372	92	464
Population in Sewerage Service Area (Person)	17,729	8,415	26,144
Wastewater Generation (MLD)	3.1	3.1	6.2

Note: "Dona Paula" includes Caranzalem area

(2) Sewer Network

Sewerage service area and proposed sewer network to be undertaken are shown in Figure 63.1.

Following two alternatives are weighed in comparison study mentioned in Volume IV Appendix M63.4 Comparison Study for Allocation of Sewerage Facilities.

Alternative 1: Wastewater will be treated at existing Tonca STP

Alternative 2: Wastewater will be treated at new STP in lowland in Taleigao

Alternative 1 was adopted, because additional STP site is not necessary and required sewer is almost same length. The existing Tonca STP has been working well and area of Tonca STP site has sufficient area for augmentation of treatment capacity.

Carrying capacities of the proposed sewers have been computed in accordance with Manning's formula. Diameter wise length of trunk sewers and branch sewers has been given in Table 63.2. The diameter of the trunk sewer (gravity) to be constructed varies from 200 mm to 500 mm to be laid through a total length of 8,300 m, and the diameter of trunk sewer (pressure) varies from 150 mm to 300 mm and its total length is 1,050 m. The branch sewers should have diameter of 150 mm stretching through a length of 55.7 km.

Flow calculation sheets for trunk sewers are shown in Volume IV Appendix M63.1 Flow Calculation Sheets (Year of 2025) and longitudinal profile of trunk sewers are drawn in Volume IV Appendix M63.2 Longitudinal Profile of Trunk Sewer (Year of 2025).

Table 63.2 Diameter wise Length of Trunk Sewer and Branch Sewer

Trunk Sewer (Gravity)

Diameter (mm)	200	250	300	350	400	450	500	Total
Length (m)	1,750	850	950	0	1,850	1,250	1,650	8,300

Trunk Line (Pressure)

Diameter (mm)	150	200	250	300	Total
Length (m)	550	0	0	600	1,150

Branch Sewer

Diameter (mm)	150	Remarks
Length (km)	55.7	464 ha, 120 m/ha

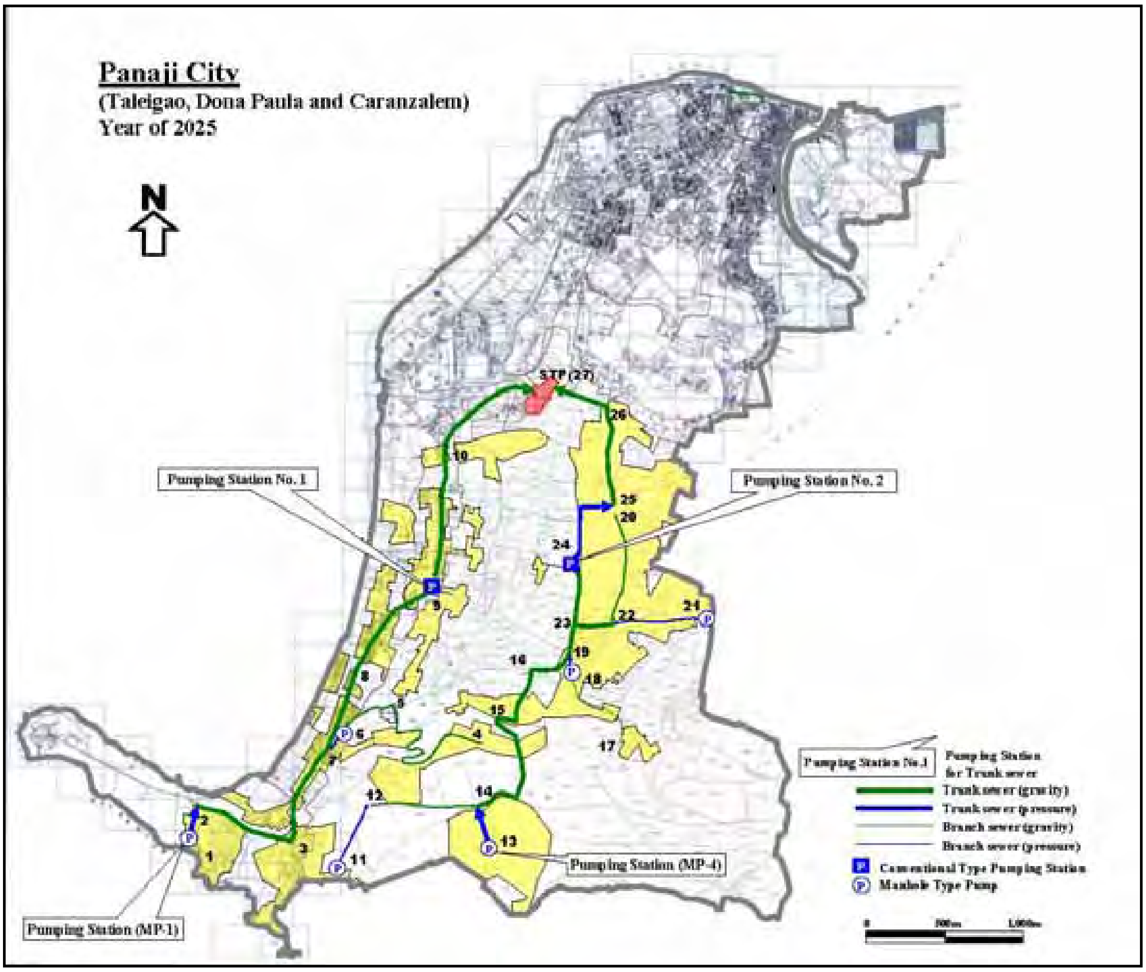


Figure 63.1 Proposed Sewerage Service Area of Panaji Surroundings

(3) Pumping Station

1) Pumping Station No. 1

Pumping Station No.1 should be located at 500 m off Miramar Beach and about 1.7 km southwest of the existing Tonca STP of Panaji City. This pumping station should receive about 80% of the wastewater from Dona Paula trunk sewer catchment area. The collected wastewater at this pumping station should be pumped up to outfall manhole at the pumping station, which finally would flow to the existing STP by gravity.

2) Pumping Station No. 2

Pumping Station No.2 should be located in lowland of Taleigao area, about 1.0 km south of the existing Tonga STP of Panaji City. This pumping station should receive about 85% of the wastewater from Taleigao trunk sewer catchment area. The collected wastewater at this pumping station should be conveyed via a 300 mm diameter pressure main about 600 m to outfall manhole, which finally would flow to the existing STP by gravity.

Another two manhole type pumping stations are proposed for sewage of trunk sewer as shown in Figure 63.1. Design flows considered for these pumping stations are presented in Table 63.3. The calculated capacities, requirement of pumps and pressure main for these pumping stations are presented in Table 63.4.

Table 63.3 Design Flows of Pumping Stations for Trunk Sewer

Pumping Station	Location (Node No.)	Design Flow (peak) (lps)	Remarks
Pumping Station No. 1	9	113.1	
Pumping Station No. 2	24	77.4	
Pumping Station (MP-1)	1	21.4	Manhole type
Pumping Station (MP-4)	13	16.1	Manhole type

Table 63.4 Design of Pumping Stations and Pressure Mains for Trunk Sewer

Pumping Station	Design Flow (peak) (lps)	Pump			Pressure Main	
		Capacity		Nos. of Pump	Diameter (mm)	Length (m)
		(lps)	(m^3/min)			
Pumping Station No. 1	113.1	29 57	1.7 3.4	2 + (0-standby) 1 + (1-standby)	N.A.	
Pumping Station No. 2	77.4	39	2.3	2 + (1-standby)	300	600
Pumping Station (MP-1)	21.4	22	1.3	1 + (1-standby)	150	50
Pumping Station (MP-4)	16.1	17	1.0	1 + (1-standby)	150	500

In addition to pumping stations mentioned above, some manhole type small pumping facilities should be installed for branch sewers in this area as shown in flow calculation sheet in Volume IV Appendix M63.1 Flow Calculation Sheets (Year of 2025) and in Figure 63.1.

(4) Sewage Treatment Plant

The Tonca STP, SBR sewage treatment plant in Panaji, was commissioned in April 2005. Sewage from Taleigao, Dona Paula, Caranzalem service blocks surrounding Panaji will be treated at Tonca STP by the following reasons.

- These three service blocks are close to Tonca STP, although lift pump stations are required in each service block.
- It is advantageous for operation and maintenance to have fewer treatment plants.
- The existing Tonca STP has enough space to construct facilities to treat additional sewage flow from these three service blocks.

1) Sewage and Sludge Treatment Method

As discussed in Section 6.2, the proposed sewage treatment plant will use the same SBR process used in the existing STP. It will also have sand filters as advanced treatment for water reuse, and a storage tank for filtered water. Sludge will be digested to reduce number of pathogen in sludge, which will make it safe for reuse. The existing sludge treatment facilities, such as sludge storage tanks, dewatering machines and drying beds, will be integrated into the proposed sludge treatment system.

2) Basic Conditions

The proposed STP facilities are designed based on the basic conditions as shown in Table 63.5.

Table 63.5 Basic Conditions for the Panaji (Tonca) STP

Item	Basic Conditions	
Design Sewage Flow	m^3/day	21,400 m^3/day
Design Sewage Quality	BOD	In: 210 mg/l, Out: 30 mg/l
	SS	In: 180 mg/l, Out: 50 mg/l
Discharge Point	Mandovi River	
Required STP Area	Not necessary (Present site have enough area for future expansion)	
Sewage Treatment	Lift pump + Screen/Grit chamber + SBR + Sand filter + Disinfection	
Sludge Treatment	Thickening + Digestion + Storage + Dewatering + (Drying Beds)	

3) Sewage Treatment Facilities

The capacity calculation based on the above basic conditions for the Panaji (Tonca) STP is shown in Volume IV Appendix M63.3 Capacity Calculation of Sewerage Treatment Facilities, the general layout plan and the summary of the proposed main facilities of STP are shown in Figures 63.2 and Table 63.6, respectively.

Table 63.6 Summary of the Proposed Main Facilities for the Panaji (Tonca) STP

Facilities	Existing Facilities (Utilization)	Additional Facilities
Wet well	15mdia×5mH×1basin	-
Pump equipment	4.35m^3/min×22kw×2units	5.20m^3/min×26kw×1unit
	9.77m^3/min×55kw×3(1)units	-
Grit chamber	4.8mW×4.8mL×1basin	-
Screen	Mechanical screen	-
SBR tank	22mW×40mL×4mH×2basins	24mW×47mL×4mH×2basins
Sand filter	-	5mW×5.5mL×4basins
Disinfection tank	2.6mW×9.5mL×1.0mH×9passes	-
Thickening tank	-	6mdia×3mH×2basins
Digestion tank	-	8mdia×8mH×2basins
Gas tank	-	9.5mdia×5mH×1basins
Dewatering	Centrifugal×20m^3/hr×2units	-
Drying beds	7.5mW×15mL×0.3mH×18basins	-

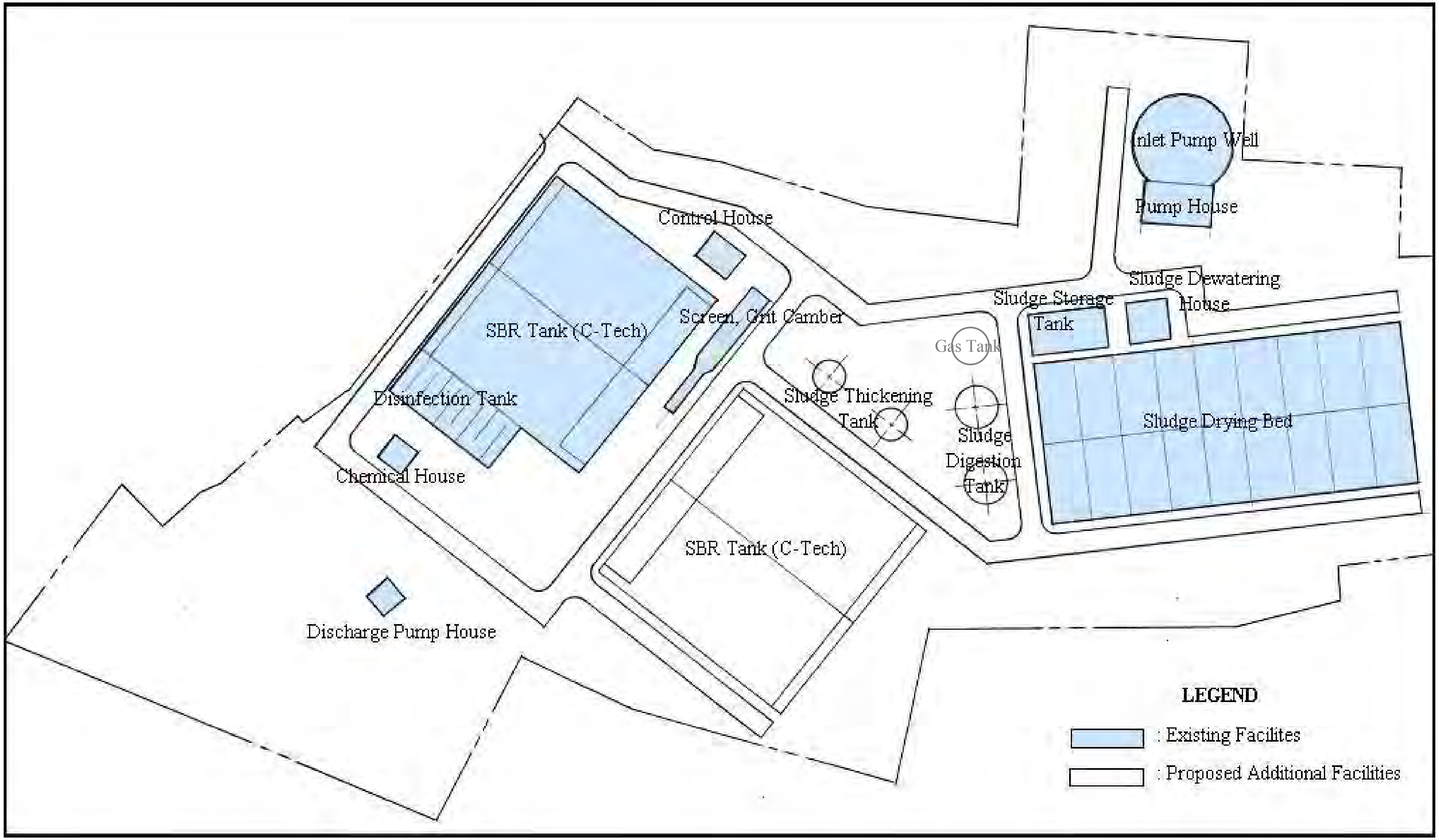


Figure 63.2 General Layout Plan of Proposed Sewage Treatment Plant in Panaji (Tonca)

6.3.2 St. Cruz

(1) General Description

This area is located in the south east of main part of Panaji City. Wastewater generated in this area should be carried to the proposed STP located at north edge of sewerage service area through a new trunk sewer with one pumping station. Covered area, population and wastewater generation under this area are shown in Table 63.7.

Table 63.7 Design Basis of St. Cruz

Item	St. Cruz
Sewerage Service Area (ha)	124
Population in Sewerage Service Area (Person)	16,918
Wastewater Generation (MLD)	2.5

(2) Sewer Network

Sewerage service area and proposed sewer network to be undertaken are shown in Figure 63.3.

Carrying capacities of the proposed sewers have been computed in accordance with Manning's formula. Diameter wise length of trunk sewers and branch sewers has been given in Table 63.8. The diameter of the trunk sewer (gravity) to be constructed varies from 200 mm to 450 mm to be laid through a total length of 2,950 m. The branch sewer should have diameter of 150 mm stretching through a length of 14.9 km.

Flow calculation sheets for trunk sewers are shown in Volume IV Appendix M63.1 Flow Calculation Sheets (Year of 2025) and longitudinal profile of trunk sewers are drawn in Volume IV Appendix M63.3 Capacity Calculation of Sewerage Treatment Facilities.

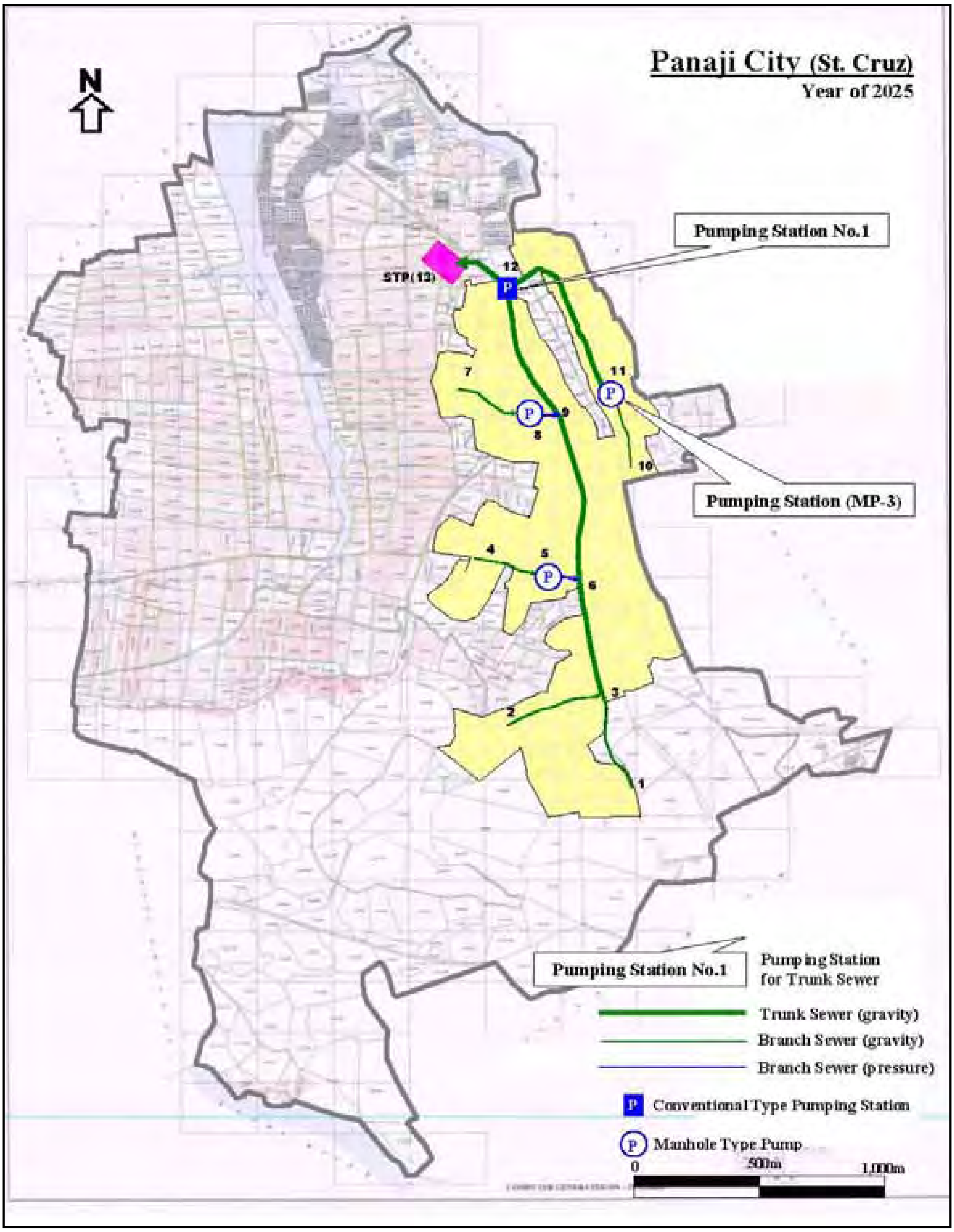


Figure 63.3 Proposed Sewerage Service Area of St. Cruz

Table 63.8 Diameter wise Length of Trunk Sewer and Branch Sewer

Trunk Sewer (Gravity)

Diameter (mm)	200	250	300	350	400	450	Total
Length (m)	750	450	0	650	600	500	2,950

Trunk Line (Pressure)

Diameter (mm)	Total
Length (m)	No pressure main

Branch Sewer

Diameter (mm)	150	Remarks
Length (km)	14.9	124 ha, 120 m/ha

(3) Pumping Station

1) Pumping Station No. 1

Pumping Station No.1 should be located at 500m east and south of proposed STP. This pumping station should be the main pumping station receiving the whole wastewater from proposed sewerage service area of St.Cruz. The collected wastewater at this pumping station should be pumped up outfall manhole at the pumping station, which finally would flow to the proposed STP by gravity.

Design flow considered for this pumping station is presented in Table 63.9. The calculated capacities, requirement of pumps and pressure main for these pumping stations are presented in Table 63.10.

Table 63.9 Design Flow of Pumping Stations for Trunk Sewer

Pumping Station	Location (Node No.)	Design Flow (peak) (lps)	Remarks
Pumping Station No. 1	12	88.1	

Table 63.10 Design of Pumping Stations and Pressure Mains for Trunk Sewer

Pumping Station	Design Flow (peak) (lps)	Pump			Pressure Main	
		Capacity		Nos. of Pump	Diameter (mm)	Length (m)
		(lps)	(m^3/min)			
Pumping Station No.1	88.1	45	2.7	2 + (1-standby)	N.A.	

In addition to one pumping station mentioned above, some manhole type small pumping facilities should be installed for branch sewer in this area as shown in flow calculation sheet in Volume IV Appendix M63.1 Flow Calculation Sheets (Year of 2025) and in Figure 63.3.

(4) Sewage Treatment Plant

The proposed STP site is a grassland for cattle grazing and does not require any resettlement. The ground level is lower than the surrounding road surface. The following strategies are used for developing layout plan of the proposed STP.

- The proposed facilities, which may release odor and/or noise will be located further away from the nearest residential area and surrounding roads.
- The ground level of the site will be raised to avoid flooding in rainy seasons.
- Landscaping will be considered for buffer zone, which surround the STP along the site boundaries in order to improve amenity in the surrounding area.

1) Sewage and Sludge Treatment Method

As discussed in Section 6.2, the proposed sewage treatment plant will use oxidation ditch process. It will also have sand filters as advanced treatment for water reuse, and a storage tank for filtered water. Sludge will be digested to reduce number of pathogen in sludge, which will make it safe for reuse.

2) Basic Conditions

The proposed STP facilities are designed based on the basic conditions as shown in Table 63.11.

Table 63.11 Basic Conditions for the St. Cruz STP

Item	Basic Conditions	
Design Sewage Flow	m^3/day	2,500 m^3/day
Design Sewage Quality	BOD	In: 300 mg/l, Out: 30 mg/l
	SS	In: 250 mg/l, Out: 50 mg/l
Discharge Point	Tributary of Mandovi River	
Required STP Area	4,000 m^2	
Sewage Treatment	Lift pump + Screen/Grit chamber + Oxidation Ditch + Final settling + Sand filter + Disinfection	
Sludge Treatment	Thickening + Digestion + Dewatering	

3) Sewage Treatment Facilities

The capacity calculation based on the basic conditions for the St.Cruz STP is shown in Volume IV Appendix M63.3 Capacity Calculation of Sewerage Treatment Facilities. The summary of the proposed main facilities of the STP is shown in Table 63.12.

Table 63.12 Summary of the Main Proposed Facilities for the St. Cruz STP

Facilities	Additional Facilities
Wet well	3.5mW×2mL×4mH×1basin
Pump equipment	$1.30m^3$/min×7kw×2units
	$2.60m^3$/min×13kw×2(1)unit
Grit chamber	0.6mW×2.9mL×0.3mH×2basins
Screen	Mechanical screen
Oxidation ditch	4mW×67mL×3mH×2basins
Final settling tank	10.5mdia×4mH×2basins
Sand filter	3mW×2.5mL×2basins
Disinfection tank	1mW×4.5mL×1mH×6passes
Thickening tank	3mdia×3mH×2basins
Digestion tank	6.5mdia×6mH×1basin
Gas tank	5.5mdia×3mH×1basins
Dewatering	Centrifugal×$2.0m^3$/hr×2(1)units

6.3.3 Porvorim

(1) General Description

This area is located in the north of main part of Panaji City on opposite bank of Mondovi River. Wastewater generated in this area should be carried to new STP near Mondovi River through a new trunk sewer with three pumping stations.

Covered area, population and wastewater generation under this area are shown in Table 63.13.

Table 63.13 Design Basis of Porvorim

Item	Main Area(1)	Riverside(2)	Total
Sewerage Service Area (ha)	523	37	560
Population in Sewerage Service Area (Person)	42,023	5,825	47,848
Wastewater Generation (MLD)	6.7	0.9	7.6

Note (1): "Main Area" is corresponding to node No.1 to 21 located on hill
Note (2): "Riverside" is corresponding to node No.22 to 26 located on riverside of branch of Mondovi River

(2) Sewer Network

Sewerage service area and proposed sewer network to be undertaken are shown in Figure 63.4.

Following two alternatives are weighed in comparison study mentioned in Volume IV Appendix M63.4 Comparison Study for Allocation of Sewerage Facilities.

Alternative 1: Wastewater will be treated at one new STP near tributary of Mandovi River

Alternative 2: Wastewater will be treated at two new STP, one STP is same as Alternative 1, and other is located in the northern part of service area

Alternative 1 was adopted because one STP is better even though trunk sewer is 6 % longer.

Carrying capacities of the proposed sewers have been computed in accordance with Manning's formula. Diameter wise length of trunk sewers and branch sewers are listed in Table 63.14. The diameter of the trunk sewer (gravity) to be constructed varies from 200 mm to 600 mm to be laid through a total length of 6,650 m, and the diameter of trunk sewer (pressure) varies from 150 mm to 350 mm and its total length is 5,000 m. The branch sewers should have diameter of 150 mm stretching through a length of 67.2 km.

Flow calculation sheets for trunk sewers are shown in Volume IV Appendix M63.1 Flow Calculation Sheets (Year of 2025) and longitudinal profile of trunk sewers are drawn in Volume IV Appendix M 63.2 Longitudinal Profile of Trunk Sewer (Year of 2025).

Table 63.14 Diameter wise Length of Trunk Sewer and Branch Sewer

Trunk Sewer (Gravity)

Diameter (mm)	200	250	300	350	400	
Length (m)	1,400	1,900	450	0	800	
Diameter (mm)	450	500	600	-	-	Total
Length (m)	700	0	1,400	-	-	6,650

Trunk Line (Pressure)

Diameter (mm)	150	200	250	300	350	Total
Length (m)	1,600	1,000	1,100	0	1,300	5,000

Branch sewer

Diameter (mm)	150	Remarks
Length (km)	67.2	560 ha, 120 m/ha

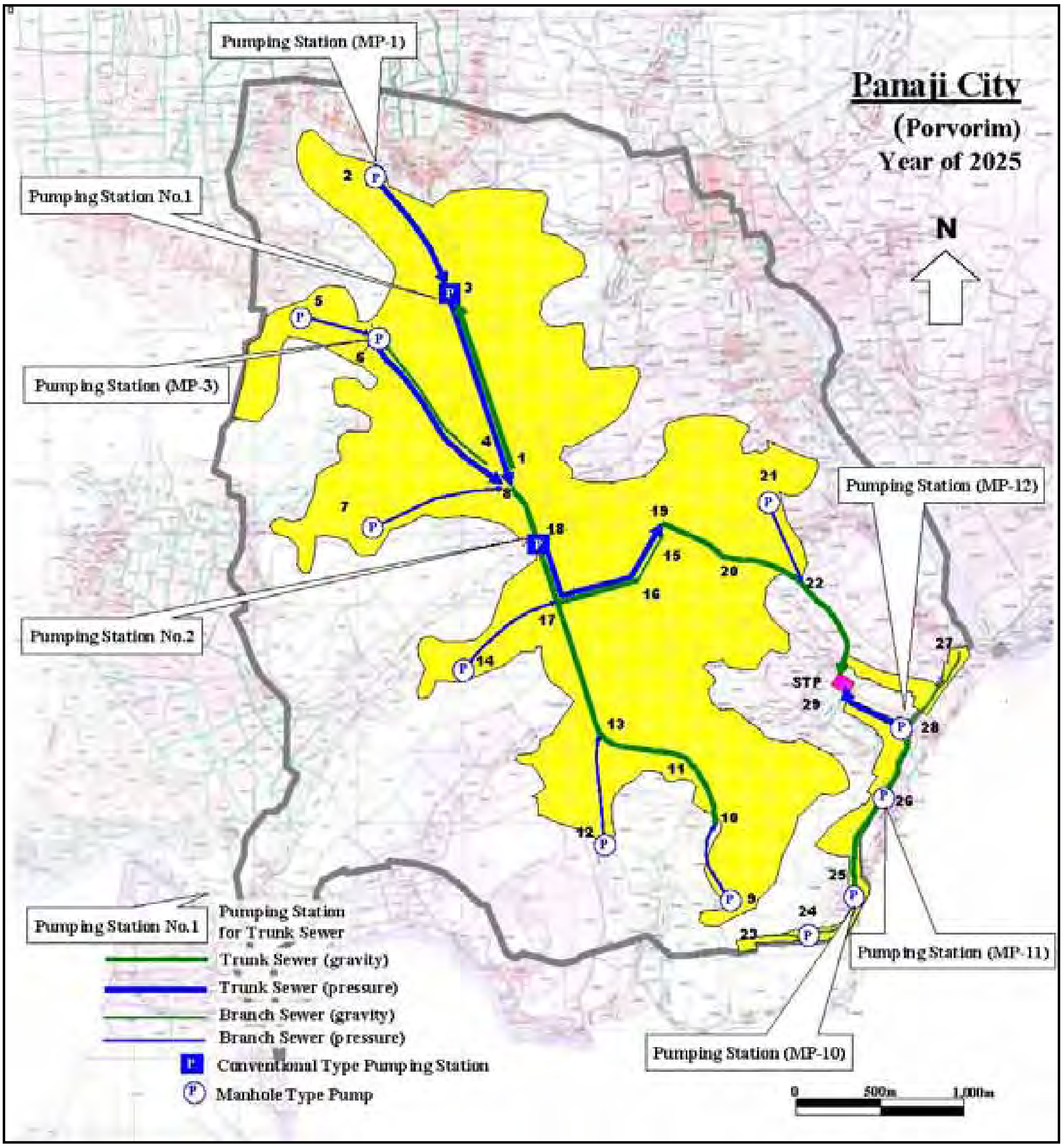


Figure 63.4 **Proposed Sewerage Service Area of Porvorim**

(3) Pumping Station

1) Pumping Station No. 1

Pumping Station No. 1 should be located along National Highway No. 17 in the north part of this area. This pumping station should receive about half of the wastewater from the north part of the sewerage service area. The collected wastewater at this pumping station should be conveyed via a 250 mm diameter pressure main about 1,100 m to outfall manhole.

2) Pumping Station No. 2

Pumping Station No. 2 should also be located along the National Highway in the center of this area. This pumping station should receive whole wastewater along National Highway No. 17 of this area. The collected wastewater should be conveyed via a 350 mm diameter pressure main about 1,300 m to outfall manhole.

Another five manhole type pumping stations are proposed for sewage of trunk sewer as shown in Figure 63.4. Design flows considered for these pumping stations are presented in Table 63.15. The calculated capacities, requirements of pumps and pressure main for these pumping stations are presented in Table 63.16.

Table 63.15 Design Flow of Pumping Stations for Trunk Sewer

Pumping Station	Location (Node No.)	Design Flow (peak) (lps)	Remarks
Pumping Station No. 1	3	73.9	
Pumping Station No. 2	18	170.2	
Pumping Station (MP-1)	2	42.6	Manhole type
Pumping Station (MP-3)	6	12.6	Manhole type
Pumping Station (MP-10)	25	13.0	Manhole type
Pumping Station (MP-11)	26	19.1	Manhole type
Pumping Station (MP-12)	28	32.2	Manhole type

Table 63.16 Design of Pumping Stations and Pressure Mains for Trunk Sewer

Pumping Station	Design Flow (peak) (lps)	Pump			Pressure Main	
		Capacity		Nos. of Pump	Diameter (mm)	Length (m)
		(lps)	(m^3/min)			
Pumping Station No. 1	73.9	37	2.2	2 + (1-standby)	250	1,100
Pumping Station No. 2	170.2	43 86	2.6 5.2	2 + (0-standby) 1 + (1-standby)	350	1,300
Pumping Station (MP-1)	42.6	43	2.6	1 + (1-standby)	200	1,000
Pumping Station (MP-3)	12.6	13	0.8	1 + (1-standby)	150	1,100
Pumping Station (MP-10)	13.0	13	0.8	1 + (1-standby)	N.A.	
Pumping Station (MP-11)	19.1	20	1.2	1 + (1-standby)	N.A.	
Pumping Station (MP-12)	32.2	33	2.0	1 + (1-standby)	150	500

In addition to pumping stations mentioned above, some manhole type small pumping stations should be installed for branch sewer in this area as shown in flow calculation sheet in Volume IV Appendix M63.1 Flow Calculation Sheets (Year of 2025) and in Figure 63.4.

(4) Sewage Treatment Plant

The proposed STP site is grassland for cattle grazing and does not require any resettlement. The ground level is lower than the surrounding road surface. The following strategies are used for developing layout plan of the proposed STP.

- The proposed facilities, which may release odor and/or noise will be located further away from the nearest residential area and surrounding roads.
- The ground level of the site will be raised to avoid flooding in rainy seasons.
- Landscaping will be considered for buffer zone, which surround the STP along the site boundaries in order to improve amenity in the surrounding area.

1) Sewage and Sludge Treatment Method

As discussed in Section 6.2, the proposed sewage treatment plant will use oxidation ditch process. It will also have sand filters as advanced treatment for water reuse, and a storage tank for filtered water. Sludge will be digested to reduce number of pathogen in sludge, which will make it safe for reuse.

2) Basic Conditions

The proposed STP facilities are designed based on the basic conditions as shown in Table 63.17.

Table 63.17 Basic Conditions for the Porvorim STP

Item	Basic Conditions	
Design Sewage Flow	m^3/day	7,600 m^3/day
Design Sewage Quality	BOD	In: 300 mg/l, Out: 30 mg/l
	SS	In: 250 mg/l, Out: 50 mg/l
Discharge Point	Tributary of Mandovi River	
Required STP Area	11,000 m^2	
Sewage Treatment	Lift pump + Screen/Grit chamber + Oxidation Ditch + Final settling + Sand filter + Disinfection	
Sludge Treatment	Thickening + Digestion + Dewatering	

3) Sewage Treatment Facilities

The capacity calculation based on the basic conditions for the Porvorim STP is shown in Volume IV Appendix M63.3 Capacity Calculation of Sewerage Treatment Facilities, the summary of the main proposed facilities of STP is shown in Table 63.18.

Table 63.18 Summary of the Main Proposed Facilities for the Porvorim STP

Facilities	Additional Facilities
Wet well	4.2mW×4mL×4mH×1basin
Pump equipment	$1.65m^3$/min×8kw×2units
	$3.30m^3$/min×16kw×1unit
	$6.60m^3$/min×31kw×2(1)unit
Grit chamber	1.3mW×3.5mL×0.3mH×2basins
Screen	Mechanical screen
Oxidation ditch	4mW×100mL×3mH×4basins
Final settling tank	13mdia×4mH×4basins
Sand filter	5mW×4mL×2basins
Disinfection tank	1.5mW×7mL×1mH×8passes
Thickening tank	5mdia×3mH×2basins
Digestion tank	10mdia×8mH×1basin
Gas tank	8mdia×4mH×1basins
Dewatering	Centrifugal×$5.0m^3$/hr×2(1)units

6.3.4 Margao

(1) General Description

Trunk sewers in northern part of Margao City (North Sewerage Zone and Central Sewerage Zone) has already been installed before year 2005 and wastewater from this area is being treated in STP constructed under the same scheme. Therefore, target sewerage service area for Margao City under this project is limited to southern part (South Sewerage Zone). Wastewater generated in this area should be carried to the existing STP through a new trunk sewer with two pumping stations. Covered area, population and wastewater generation of this area are shown in Table 63.19.

Table 63.19 Design Basis of Margao City

Item	North & Central(1)	South(2)	Total
Sewerage Service Area (ha)	548	511	1,059
Population in Sewerage Service Area (Person)	61,286	56,907	118,193
Wastewater Generation (MLD)	10.9	10.0	20.9

Note (1): "North & Central" is corresponding to "North and Central Sewerage Zone", existing
Note (2): "South" is South Sewerage Zone proposed in this Master Plan scheme

(2) Sewer Network

Sewerage service area and proposed sewer network to be undertaken are shown in Figure 63.5.

Following two alternatives are weighed in comparison study mentioned in Volume IV Appendix M63.4 Comparison Study for Allocation of Sewerage Facilities.

Alternative1: Wastewater will be treated at existing Margao STP
Railway crossing point is west end of service area

Alternative 2: Wastewater will be treated at existing Margao STP (same as Alternative 1)
Railway crossing point is about center of service area

Alternative 1 was adopted because shorter trunk sewer is necessary and construction of sewer at railway crossing is easier although main route diverts along the municipal boundary.

Carrying capacities of the proposed sewers have been computed in accordance with Manning's formula. Diameter wise length of trunk sewers and branch sewers are listed in Table 63.20. The diameter of the trunk sewer (gravity) to be constructed varies from 200 mm to 700 mm to be laid through a total length of 7,900m, and the diameter of trunk sewer (pressure) varies from 150 mm to 400 mm and its total length is 1,100 m. The branch sewers should have diameter of 150 mm stretching through a length of 50.8 km.

Flow calculation sheets for trunk sewers are shown in Volume IV Appendix M63.1 Flow Calculation Sheets (Year of 2025) and longitudinal profile of trunk sewers are drawn in Volume IV Appendix M 63.2 Longitudinal Profile of Trunk Sewer (Year of 2025).

Table 63.20 Diameter wise Length of Trunk Sewer and Branch Sewer

Trunk Sewer (Gravity)

Diameter (mm)	200	250	300	350	400	
Length (m)	300	600	500	1,200	750	
Diameter (mm)	450	500	600	700	-	Total
Length (m)	650	600	0	3,300	-	7,900

Trunk Line (Pressure)

Diameter (mm)	150	200	250	300	350	400	Total
Length (m)	450	0	0	0	0	650	1,100

Branch sewer

Diameter (mm)	150	Remarks
Length (km)	50.8	254 ha, 200 m/ha

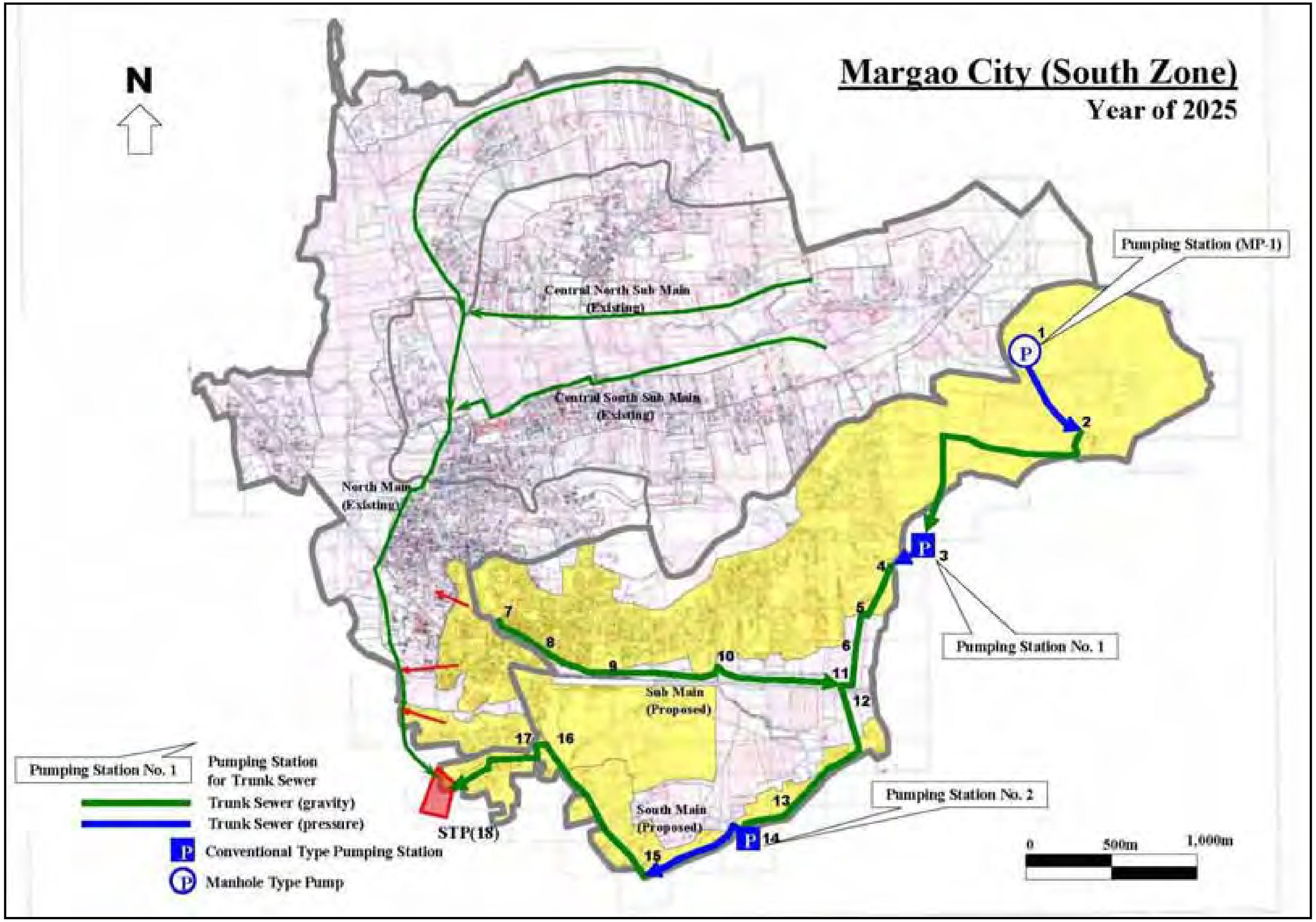


Figure 63.5 Proposed Sewerage Service Area of Margao City

(3) Pumping Station

1) Pumping Station No. 1

Pumping Station No.1 should be located near PWD branch office at east end of the City. This pumping station should receive wastewater from newly developing residential colonies in the most eastern part of the City. The collected wastewater should be pumped up to outfall manhole at the pumping station, which should flow to manhole of trunk sewer by gravity.

2) Pumping Station No. 2

Pumping Station No.2 should be located at south end of the City, 2,400 m to existing STP along proposed trunk sewer. This pumping station should receive about 80% of the wastewater from target area (South Sewerage Zone) including main part of the City. The collected wastewater should be conveyed via a 400 mm diameter pressure main about 650 m to outfall manhole at ridge, which should finally flow to the existing STP.

Another one manhole type pumping stations are proposed for sewage of trunk sewer as shown in Figure 63.5. Design flows considered for these pumping stations are presented in Table 63.21. The calculated capacities, requirements of pumps and pressure main for these pumping stations are presented in Table 63.22.

Table 63.21 Design Flows of Pumping Stations for Trunk Sewer

Pumping Station	Location (Node No.)	Design Flow (peak) (lps)	Remarks
Pumping Station No. 1	3	81.3	
Pumping Station No. 2	14	231.1	
Pumping Station (MP-1)	1	28.6	Manhole type

Table 63.22 Design of Pumping Stations and Pressure Mains for Trunk Sewer

Pumping Station	Design Flow (peak) (lps)	Pump			Pressure Main	
		Capacity		Nos. of Pump	Diameter (mm)	Length (m)
		(lps)	(m^3/min)			
Pumping Station No. 1	81.3	41	2.5	2 + (1-standby)	N.A.	
Pumping Station No. 2	231.1	29 58 116	1.7 3.5 7.0	2 + (0-standby) 1 + (0-standby) 1 + (1-standby)	400	650
Pumping Station (MP-1)	28.6	29	1.7	1 + (1-standby)	150	450

In addition to pumping stations mentioned above, some manhole type small pumping stations should be implemented for branch sewer in this area.

(4) Sewage Treatment Plant

The existing Margao STP that used a conventional activated sludge process was commissioned in May 2000. The proposed extension of the STP that will treat the projected sewage flow in 2025 is designed to fit into the existing treatment plant site.

1) Sewage and Sludge Treatment Method

As discussed in Section 6.2, the proposed sewage treatment plant will use the same conventional activated sludge process used in the existing STP. Although the existing STP does not have a disinfection facility, the proposed facility will have a disinfection facility to treat the whole incoming sewage flow in order to make effluent safe for the receiving water. It will also have sand filters as advanced treatment for water reuse, and a storage tank for filtered water. Sludge will be digested to reduce number of pathogen in sludge, which will make it safe for reuse. The existing sludge treatment facilities, such as sludge digesters and drying beds, will be integrated into the proposed sludge treatment system.

2) Basic Conditions

The proposed STP facilities are designed based on the basic conditions as shown in Table 63.23.

Table 63.23 Basic Conditions for the Margao STP

Item	Basic Conditions	
Design Sewage Flow	m^3/day	20,900 m^3/day
Design Sewage Quality	BOD	In: 300 mg/l, Out: 30 mg/l
	SS	In: 250 mg/l, Out: 50 mg/l
Discharge Point	Sal River	
Required STP Area	No (Present site have enough area for future expansion)	
Sewage Treatment	Lift pump + Screen/Grit chamber + Primary settling + Aeration + Secondary settling + Sand filter + Disinfection	
Sludge Treatment	Thickening + Digestion + Storage + Dewatering + (Drying Beds)	

3) Sewage Treatment Facilities

The capacity calculation based on the basic conditions for the Margao STP is shown in Volume IV Appendix M63.3 Capacity Calculation of Sewerage Treatment Facilities, the general layout plan and the summary of the proposed main facilities of STP are shown in Figure 63.6 and Table 63.24, respectively.

Table 63.24 Summary of the Proposed Main Facilities for the Margao STP

Facilities	Existing Facilities (Utilization)	Additional Facilities
Wet well	12.3mdia×15mH×1basin	-
Pump equipment	$3.33m^3$/min×25HP×2units	$12.68m^3$/min×48kw×2(1)1units
	$6.66m^3$/min×50HP×2units	-
Grit chamber	1.25mW×8mL×0.35mH×1basin	1.3mW×9.5mL×0.35mH×1basin
Screen	Mechanical screen	-
Primary settling	18mdia×3mH×1basin	16.5mdia×3mH×1basin
Aeration tank	12mW×33mL×3mH×1basin	12mW×24mL×4mH×2basins
Secondary settling	21mdia×3mH×1basin	18mdia×4mH×2basins
Sand filter	-	5mW×5.5mL×4basins
Disinfection	-	2mW×10.5mL×1.2mH×9passes
Thickening tank	-	8.5mdia×3mH×2basins
Digestion tank	18mdia×10.65mH×1basin	-
Gas tank	-	11.5mdia×5mH×1basins
Dewatering	-	Centrifugal×$14m^3$/hr×2(1)units
Drying beds	12.4mW×12.8mL×0.3mH×14basins	12mW×10.5mL×0.3mH×10basins

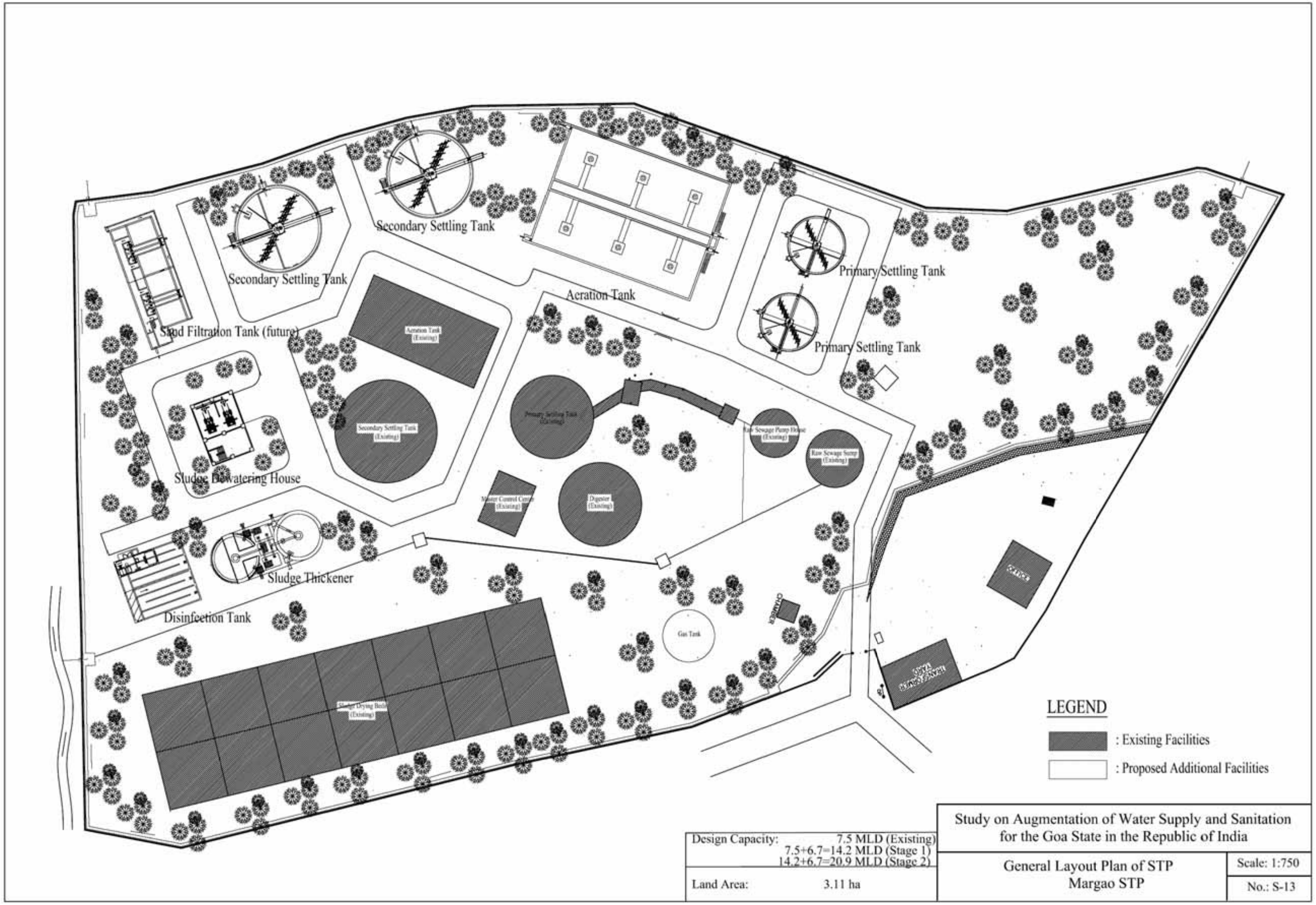


Figure 63.6 General Layout Plan of Proposed Sewage Treatment Plant in Margao

6.3.5 Ponda

(1) General Description

This area covers almost all settlement area of Ponda City. Wastewater generated in this area should be carried to new STP near main drainage, just downstream of the City, through two new trunk sewers on both sides of the drainage with one pumping station. Covered area, population and wastewater generation of this area are shown in Table 63.25.

Table 63.25 Design Basis of Ponda City

Item	Ponda
Sewerage Service Area (ha)	201
Population in Sewerage Service Area (Person)	19,401
Wastewater Generation (MLD)	3.5

(2) Sewer Network

Sewerage service area and proposed sewer network to be undertaken are shown in Figure 63.7.

Following two alternatives are weighed in comparison study mentioned in Volume IV Appendix M63.4 Comparison Study for Allocation of Sewerage Facilities.

Alternative1: Wastewater will be treated at new STP at down stream of service area
Trunk sewer will be laid under main road running parallel to main drainage

Alternative 2: Wastewater will be treated at same new STP as Alternative 1
Trunk sewer will be laid in main drainage area or very close to the drainage

Alternative 1 was adopted because it is impossible to lay trunk sewer in drainage area or close to the drainage due to many residential facing or close to the drainage

Carrying capacities of the proposed sewers have been computed in accordance with Manning's formula. Diameter wise length of trunk sewers and branch sewers are listed in Table 63.26. The diameter of the trunk sewer (gravity) to be constructed varies from 200 mm to 450 mm to be laid through a total length of 2,650 m, and the diameter of trunk sewer (pressure) varies from 150 mm to 250 mm and its total length is 750 m. The branch sewer should have diameter of 150 mm stretching through a length of 24.1 km.

Flow calculation sheets for trunk sewers are shown in Volume IV Appendix M63.1 Flow Calculation Sheets (Year of 2025) and longitudinal profile of trunk sewers are drawn in Volume IV Appendix M 63.2 Longitudinal Profile of Trunk Sewer (Year of 2025).

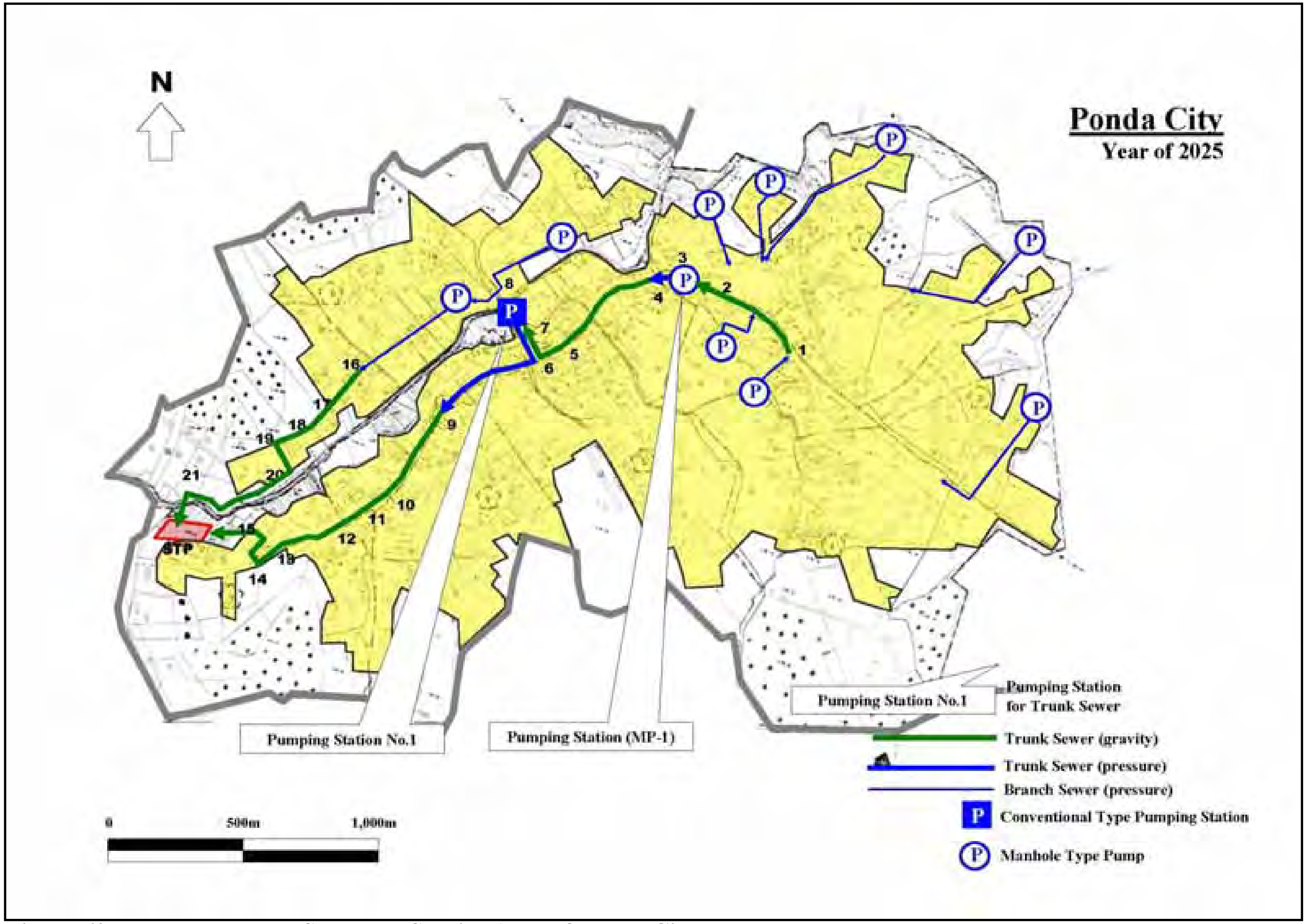


Figure 63.7 Proposed Sewerage Service Area of Ponda City

Table 63.26 Diameter wise Length of Trunk Sewer and Branch Sewer

Trunk Sewer (Gravity)

Diameter (mm)	200	250	300	350	400	450	Total
Length (m)	0	950	50	200	950	500	2,650

Trunk Line (Pressure)

Diameter (mm)	150	200	250	Total
Length (m)	50	250	450	750

Branch Sewer

Diameter (mm)	150	Remarks
Length (km)	24.1	201 ha, 120 m/ha

(3) Pumping Station

1) Pumping Station No. 1

Pumping Station No.1 should be located on left bank of the main drainage, near the main bus terminal at the center of the City. This pumping station should receive about half of the wastewater from proposed sewerage service area of the City. The collected wastewater at this pumping station should be conveyed via a 250 mm diameter pressure main about 450 m to outfall manhole, which should finally flow to the STP by gravity.

Another one manhole type pumping stations are proposed for sewage of trunk sewer as shown in Figure 63.7. Design flows considered for this pumping station are presented in Table 63.27. The calculated capacities, requirements of pumps and pressure main for this pumping station are presented in Table 63.28.

Table 63.27 Design Flow of Pumping Stations for Trunk Sewer

Pumping Station	Location (Node No.)	Design Flow (peak) (lps)	Remarks
Pumping Station No. 1	8	69.6	
Pumping Station (MP-1)	3	49.0	Manhole type

Table 63.28 Design of Pumping Stations and Pressure Mains for Trunk Sewer

Pumping Station	Design Flow (peak) (lps)	Pump			Pressure Main	
		Capacity		Nos. of Pump	Diameter (mm)	Length (m)
		(lps)	(m^3/min)			
Pumping Station No. 1	69.6	35	2.1	2 + (1-standby)	250	450
Pumping Station (MP-1)	49.0	49	2.9	1 + (1-standby)	200	250

In addition to pumping stations mentioned above, some manhole type small pumping stations should be implemented for branch sewer in this area as shown in Figure 63.7.

(4) Sewage Treatment Plant

The proposed STP site is a grassland for cattle grazing and does not require any resettlement. The ground level is lower than the surrounding road surface. The following strategies are used for developing layout plan of the proposed STP.

- The proposed facilities, which may release odor and/or noise will be located further away from the nearest residential area and surrounding roads.
- The ground level of the site will be raised to avoid flooding in rainy seasons.
- Landscaping will be considered for buffer zone, which surround the STP along the site boundaries in order to improve amenity in the surrounding area.

1) Sewage and Sludge Treatment Method

As discussed in Section 6.2, the proposed sewage treatment plant will use oxidation ditch process. It will also have sand filters as advanced treatment for water reuse, and a storage tank for filtered water. Sludge will be digested to reduce number of pathogen in sludge, which will make it safe for reuse.

2) Basic Conditions

The proposed STP facilities are designed based on the basic conditions as shown in Table 63.29.

Table 63.29 Basic Conditions for the Ponda STP

Item	Basic Conditions	
Design Sewage Flow	m^3/day	3,500 m^3/day
Design Sewage Quality	BOD	In: 280 mg/l, Out: 30 mg/l
	SS	In: 240 mg/l, Out: 50 mg/l
Discharge Point	Tributary of Zuari River	
Required STP Area	5,300 m^2	
Sewage Treatment	Lift pump + Screen/Grit chamber + Oxidation Ditch + Final settling + Sand filter + Disinfection	
Sludge Treatment	Thickening + Digestion + Dewatering	

3) Sewage Treatment Facilities

The capacity calculation based on the basic conditions for the Ponda STP is shown in Volume IV Appendix M63.3 Capacity Calculation of Sewerage Treatment Facilities, the summary of the main proposed facilities of STP is shown in Table 63.30.

Table 63.30 Summary of the Main Proposed Facilities for the Ponda STP

Facilities	Additional Facilities
Wet well	4mW×2.5mL×4mH×1basin
Pump equipment	$1.82m^3$/min×9kw×2units
	$3.65m^3$/min×18kw×2(1)unit
Grit chamber	0.7mW×3.5mL×0.3mH×2basins
Screen	Mechanical screen
Oxidation ditch	4mW×92mL×3mH×2basins
Final settling tank	12.5mdia×4mH×4basins
Sand filter	3mW×3mL×2basins
Disinfection tank	1mW×5mL×1mH×8passes
Thickening tank	3.5mdia×3mH×2basins
Digestion tank	6mdia×8mH×1basin
Gas tank	6mdia×3mH×1basins
Dewatering	Centrifugal×$2.5m^3$/hr×2(1)units

6.3.6 Mapusa

(1) General Description

This area covers almost all settlement area of Mapusa City. Wastewater generated in this area should be carried to new STP near main drainage at east edge of the City through two new trunk sewers with one pumping station. Covered area, population and wastewater generation of this area are shown in Table 63.31.

Table 63.31 Design Basis of Mapusa City

Item	Mapusa
Sewerage Service Area (ha)	322
Population in Sewerage Service Area (Parson)	68,255
Wastewater Generation (MLD)	10.8

(2) Sewer Network

Sewerage service area and proposed sewer network to be undertaken are shown in Figure 63.8.

Following two alternatives are weighed in comparison study mentioned in Volume IV Appendix M63.4 Comparison Study for Allocation of Sewerage Facilities.

Alternative 1: Wastewater will be treated at new STP at east edge of service area

Alternative 2: Wastewater will be treated at new STP at south of Municipal Market

Alternative 1 was adopted because of shorter trunk sewer and less number of pumping station, although Alternative 2 is easier to collect densely area.

Carrying capacities of the proposed sewers have been computed in accordance with Manning's formula. Diameter wise length of trunk sewers and branch sewers are listed in Table 63.32. The diameter of the trunk sewer (gravity) to be constructed varies from 250 mm to 700 mm to be laid through a total length of 7,750 m, and the diameter of trunk sewer (pressure) is 300 mm and its total length is 950 m. The branch sewers should have diameter of 150 mm stretching through a length of 47.0 km.

Flow calculation sheets for trunk sewers are shown in Volume IV Appendix M63.1 Flow Calculation Sheets (Year of 2025) and longitudinal profile of trunk sewers are drawn in Volume IV Appendix M63.2 Longitudinal Profile of Trunk Sewer (Year of 2025).

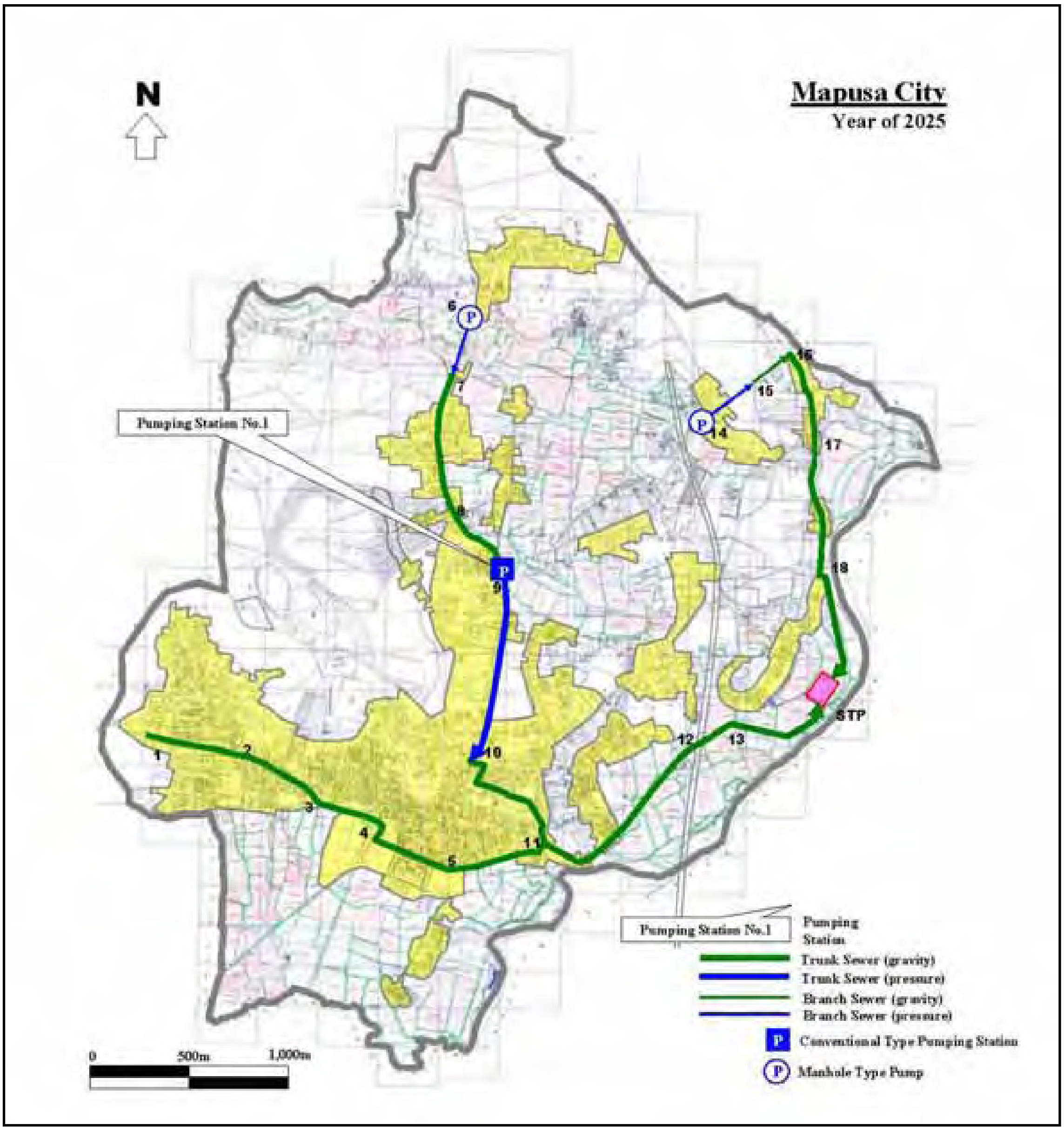


Figure 63.8 **Proposed Sewerage Service Area of Mapusa City**

Table 63.32　　Diameter wise Length of Trunk Sewer and Branch Sewer

Trunk Sewer (gravity)

Diameter (mm)	200	250	300	350	400	
Length (m)	0	1,900	1,550	0	0	
Diameter (mm)	450	500	600	700	-	Total
Length (m)	1,700	400	400	1,800	-	7,750

Trunk Line (pressure)

Diameter (mm)	300	Total
Length (m)	950	950

Branch sewer

Diameter (mm)	150	Remarks
Length (km)	47.0	124 ha, 200 m/ha (Congested Area) 185 ha, 120 m/ha (Other Area)

(3)　　Pumping Station

1)　　Pumping Station No. 1

Pumping Station No.1 should be located in lowland at center of the City.　This pumping station should receive almost all wastewater from the north of the ridge line.　The collected wastewater should be conveyed via a 300 mm diameter pressure main about 950 m to outfall manhole, which should finally flow to STP by gravity.

Design flows considered for this pumping station are presented in Table 63.33.　The calculated capacities and requirements of pumps and pressure main for this pumping station are presented in Table 63.34.

Table 63.33　　Design Flow of Pumping Stations for Trunk Sewer

Pumping Station	Location (Node No.)	Design Flow (peak) (lps)	Remarks
Pumping Station (PS-1)	9	102.7	

Table 63.34　　Design of Pumping Stations and Pressure Mains for Trunk Sewer

Pumping Station	Design Flow (peak) (lps)	Pump			Pressure Main	
		Capacity		Nos. of Pump	Diameter (mm)	Length (m)
		(lps)	(m^3/min)			
Pumping Station (PS-1)	102.7	26 52	1.6 3.1	2 + (0-standby) 1 + (1-standby)	300	950

In addition to pumping stations mentioned above, some manhole type small pumping stations should be implemented for branch sewer in this area as shown in Figure 63.8.

(4) Sewage Treatment Plant

The proposed STP site is grassland for cattle grazing and does not require any resettlement. The ground level is lower than the surrounding road surface. The following strategies are used for developing layout plan of the proposed STP.

- The proposed facilities, which may release odor and/or noise will be located further away from the nearest residential area and surrounding roads.
- The ground level of the site will be raised to avoid flooding in rainy seasons.
- Landscaping will be considered for buffer zone, which surround the STP along the site boundaries in order to improve amenity in the surrounding area.

1) Sewage and Sludge Treatment Method

As discussed in Section 6.2, the proposed sewage treatment plant will use oxidation ditch process. It will also have sand filters as advanced treatment for water reuse, and a storage tank for filtered water. Sludge will be digested to reduce number of pathogen in sludge, which will make it safe for reuse.

2) Basic Conditions

The proposed STP facilities are designed based on the basic conditions as shown in Table 63.35.

Table 63.35 Basic Conditions for the Mapusa STP

Item	Basic Conditions	
Design Sewage Flow	m^3/day	10,800 m^3/day
Design Sewage Quality	BOD	In: 300 mg/l, Out: 30 mg/l
	SS	In: 250 mg/l, Out: 50 mg/l
Discharge Point	Tributary of Mandovi River	
Required STP Area	15,500 m^2	
Sewage Treatment	Lift pump + Screen/Grit chamber + Oxidation Ditch + Final settling + Sand filter + Disinfection	
Sludge Treatment	Thickening + Digestion + Dewatering	

3) Sewage Treatment Facilities

The capacity calculation based on the basic conditions for the Mapusa STP is shown in Volume IV Appendix M63.3 Capacity Calculation of Sewerage Treatment Facilities, the summary of the main proposed facilities of STP is shown in Table 63.36.

Table 63.36 Summary of the Main Proposed Facilities for the Mapusa STP

Facilities	Additional Facilities
Wet well	4.5mW×4mL×5mH×1basin
Pump equipment	$2.11m^3$/min×10kw×2units
	$4.22m^3$/min×20kw×1unit
	$8.44m^3$/min×40kw×2(1)units
Grit chamber	1.6mW×3.6mL×0.3mH×2basins
Screen	Mechanical screen
Oxidation ditch	4mW×142mL×3mH×4basins
Final settling tank	15.5mdia×4mH×4basins
Sand filter	5mW×5.5mL×2basins
Disinfection tank	2mW×10mL×1mH×6passes
Thickening tank	6mdia×3mH×2basins
Digestion tank	8mdia×8mH×2basins
Gas tank	12mdia×5mH×1basins
Dewatering	Centrifugal×$7.0m^3$/hr×2(1)units

6.3.7 South Coastal Belt

(1) General Description

This area covers almost all settlement area of Colva Village, a part of South Coastal Belt. Wastewater generated in this area should be carried to new STP at south end of the Village through new trunk sewer with one pumping station. Treated water should be conveyed by pressure main to Sal River, about 3 km from the STP. Covered area, population and wastewater generation of this area are shown in Table 63.37.

Table 63.37 Design Basis of South Coastal Belt

Item	Colva
Sewerage Service Area (ha)	110
Population in Sewerage Service Area (Person)	5,279
Wastewater Generation (MLD)	2.2

(2) Sewer Network

Sewerage service area and proposed sewer network to be undertaken are shown in Figure 63.9.

Following two alternatives are weighed in comparison study mentioned in Volume IV Appendix M 63.4 Comparison Study for Allocation of Sewerage Facilities.

Alternative 1: Wastewater will be treated at new STP at south edge of service area

Alternative 2: Wastewater will be treated at new STP at north edge of service area

Alternative 1 was adopted because of shorter trunk sewer and shorter effluent discharge line to Sal river than Alternative 2.

Carrying capacities of the proposed sewers have been computed in accordance with Manning's formula. Diameter wise length of trunk sewers and branch sewers are listed in Table 63.38. The diameter of the trunk sewer (gravity) to be constructed varies from 200 mm to 400 mm to be laid through a total length of 2,850 m, and the diameter of trunk sewer (pressure) is 150 mm and its total length is 800 m. The branch sewer should have diameter of 150 mm stretching through a length of 13.2 km.

Flow calculation sheets for trunk sewers are shown in Volume IV Appendix M63.1 Flow Calculation Sheets (Year of 2025) and longitudinal profile of trunk sewers are drawn in Volume IV Appendix M63.2 Longitudinal Profile of Trunk Sewer (Year of 2025).

Table 63.38 Diameter wise Length of Trunk Sewer and Branch Sewer

Trunk Sewer (Gravity)

Diameter (mm)	200	250	300	350	400	Total
Length (m)	1,000	350	1,000	0	500	2,850

Trunk Line (Pressure)

Diameter (mm)	150	Total
Length (m)	800	800

Branch Sewer

Diameter (mm)	150	Remarks
Length (km)	13.2	110 ha, 120 m/ha

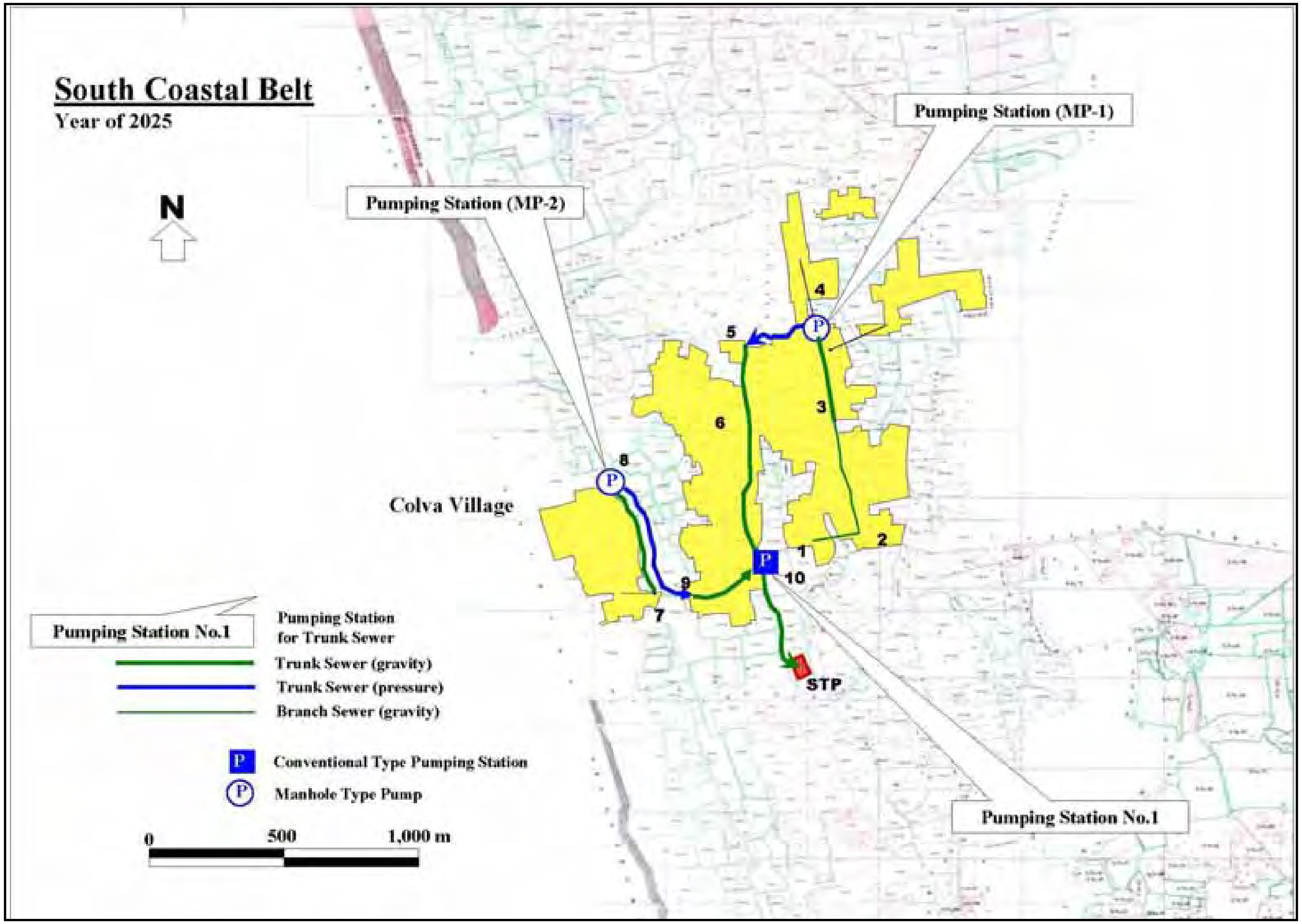


Figure 63.9 Proposed Sewerage Service Area of South Coastal Belt

(3) Pumping Station

1) Pumping Station No. 1

Pumping Station No.1 should be located at the center of the sewerage service area. This pumping station should receive whole wastewater of the area. The collected wastewater should be pumped to outfall manhole at the pumping station, which should flow to the proposed STP by gravity.

Another two manhole type pumping stations are proposed for sewage of trunk sewer as shown in Figure 63.9. Design flows considered for this pumping station are presented in Table 63.39. The calculated capacities, requirements of pumps and pressure main for this pumping station are presented in Table 63.40.

Table 63.39 Design Flows of Pumping Stations

Pumping Station	Location (Node No.)	Design Flow (peak) (lps)	Remarks
Pumping Station No. 1	10	74.8	
Pumping Station (MP-1)	4	27.2	Manhole type
Pumping Station (MP-2)	8	17.0	Manhole type

Table 63.40 Design of Pumping Stations and Pressure Mains

Pumping Station	Design Flow (peak) (lps)	Pump			Pressure Main	
		Capacity		Nos. of Pump	Diameter (mm)	Length (m)
		(lps)	(m^3/min)			
Pumping Station No.1	74.8	38	2.3	2 + (1-standby)	N.A.	
Pumping Station (MP-1)	27.2	28	1.7	1 + (1-standby)	150	300
Pumping Station (MP-2)	17.0	17	1.0	1 + (1-standby)	150	500

In addition to pumping stations mentioned above, some manhole type small pumping stations should be implemented for branch sewer in this area.

(4) Sewage Treatment Plant (Colva STP)

The proposed STP site is a grassland for cattle grazing and does not require any resettlement. The ground level is lower than the surrounding road surface. The following strategies are used for developing layout plan of the proposed STP.

- The proposed facilities, which may release odor and/or noise will be located further away from the nearest residential area and surrounding roads.
- The ground level of the site will be raised to avoid flooding in rainy seasons.

- Landscaping will be considered for buffer zone, which surround the STP along the site boundaries in order to improve amenity in the surrounding area.

1) Sewage and Sludge Treatment Method

As discussed in Section 6.2, the proposed sewage treatment plant will use oxidation ditch process. It will also have sand filters as advanced treatment for water reuse, and a storage tank for filtered water. Sludge will be digested to reduce number of pathogen in sludge, which will make it safe for reuse.

2) Basic Conditions

The proposed STP facilities are designed based on the basic conditions as shown in Table 63.41.

Table 63.41 Basic Conditions for the Colva STP

Item	Basic Conditions	
Design Sewage Flow	m^3/day	2,200 m^3/day
Design Sewage Quality	BOD	In: 220 mg/l, Out: 30 mg/l
	SS	In: 190 mg/l, Out: 50 mg/l
Discharge Point	Sal River	
Required STP Area	3,500 m^2	
Sewage Treatment	Lift pump + Screen/Grit chamber + Oxidation Ditch + Final settling + Sand filter + Disinfection	
Sludge Treatment	Thickening + Digestion + Dewatering	

3) Sewage Treatment Facilities

The capacity calculation based on the basic conditions for the Colva STP is shown in Volume IV Appendix M63.3 Capacity Calculation of Sewerage Treatment Facilities, the summary of the main proposed facilities of STP is shown in Table 63.42.

Table 63.42 Summary of the Main Proposed Facilities for the Colva STP

Facilities	Additional Facilities
Wet well	3.5mW×1.9mL×3.5mH×1basin
Pump equipment	$1.15m^3$/min×6kw×2units
	$2.29m^3$/min×11kw×2(1)units
Grit chamber	0.6mW×2.6mL×0.3mH×2basins
Screen	Mechanical screen
Oxidation ditch	4mW×59mL×3mH×2basins
Final settling tank	10mdia×4mH×2basins
Sand filter	4mW×2mL×2basins
Disinfection tank	1mW×4mL×1mH×6passes
Thickening tank	3mdia×3mH×1basin
Digestion tank	5mdia×6mH×1basin
Gas tank	4mdia×3mH×1basins
Dewatering	Centrifugal×$1.0m^3$/hr×2(1)units

6.3.8 North Coastal Belt

(1) General Description

This area contains Candolim Village and Calangute Village. Wastewater generated in this area should be carried to new STP at north edge of Calangute Village near Baga River bank through new trunk sewers with two pumping stations. Covered area, population and wastewater generation under these two villages are shown in Table 63.43.

Table 63.43 Design Basis of North Coastal Belt

Item	Candolim	Calangute	Total
Sewerage Service Area (ha)	200	425	625
Population in Sewerage Service Area (Person)	13,224	26,130	39,354
Wastewater Generation (MLD)	4.1	7.1	11.2

(2) Sewer Network

Sewerage service area and proposed sewer network to be undertaken are shown in Figure 63.10.

Following two alternatives are weighed in comparison study mentioned in Volume IV Appendix M63.4 Comparison Study for Allocation of Sewerage Facilities.

Alternative 1: Wastewater will be treated at one new STP at north edge of Calangute

Alternative 2: Wastewater will be treated at two new STP, one is same as Alternative 1, and other is located at south end of Candolim

Alternative 1 was adopted, because high treatment reliability will be realized by centralized one STP.

Carrying capacities of the proposed sewers have been computed in accordance with Manning's formula. Diameter wise length of trunk sewers and branch sewers are listed in Table 63.44. The diameter of the trunk sewer (gravity) to be constructed varies from 250 mm to 800 mm to be laid through a total length of 9,900 m, and the diameter of trunk sewer (pressure) varies from 200 mm to 400 mm, and its total length is 4,200 m. The branch sewers should have diameter of 150 mm stretching through a length of 71.2 km.

Flow calculation sheets for trunk sewers are shown in Volume IV Appendix M63.1 Flow Calculation Sheets (Year of 2025) and longitudinal profile of trunk sewers are drawn in Volume IV Appendix M63.2 Longitudinal Profile of Trunk Sewer (Year of 2025).

Table 63.44 Diameter wise Length of Trunk Sewer and Branch Sewer

Trunk Sewer (Gravity)

Diameter (mm)	200	250	300	350	400	
Length (m)	0	700	1,300	1,100	0	
Diameter (mm)	450	500	600	700	800	Total
Length (m)	1,400	700	400	2,500	1,800	9,900

Trunk Line (Pressure)

Diameter (mm)	200	250	300	350	400	Total
Length (m)	1,200	2,500	0	0	500	4,200

Branch sewer

Diameter (mm)	150	Remarks
Length (km)	71.2	593 ha, 120 m/ha

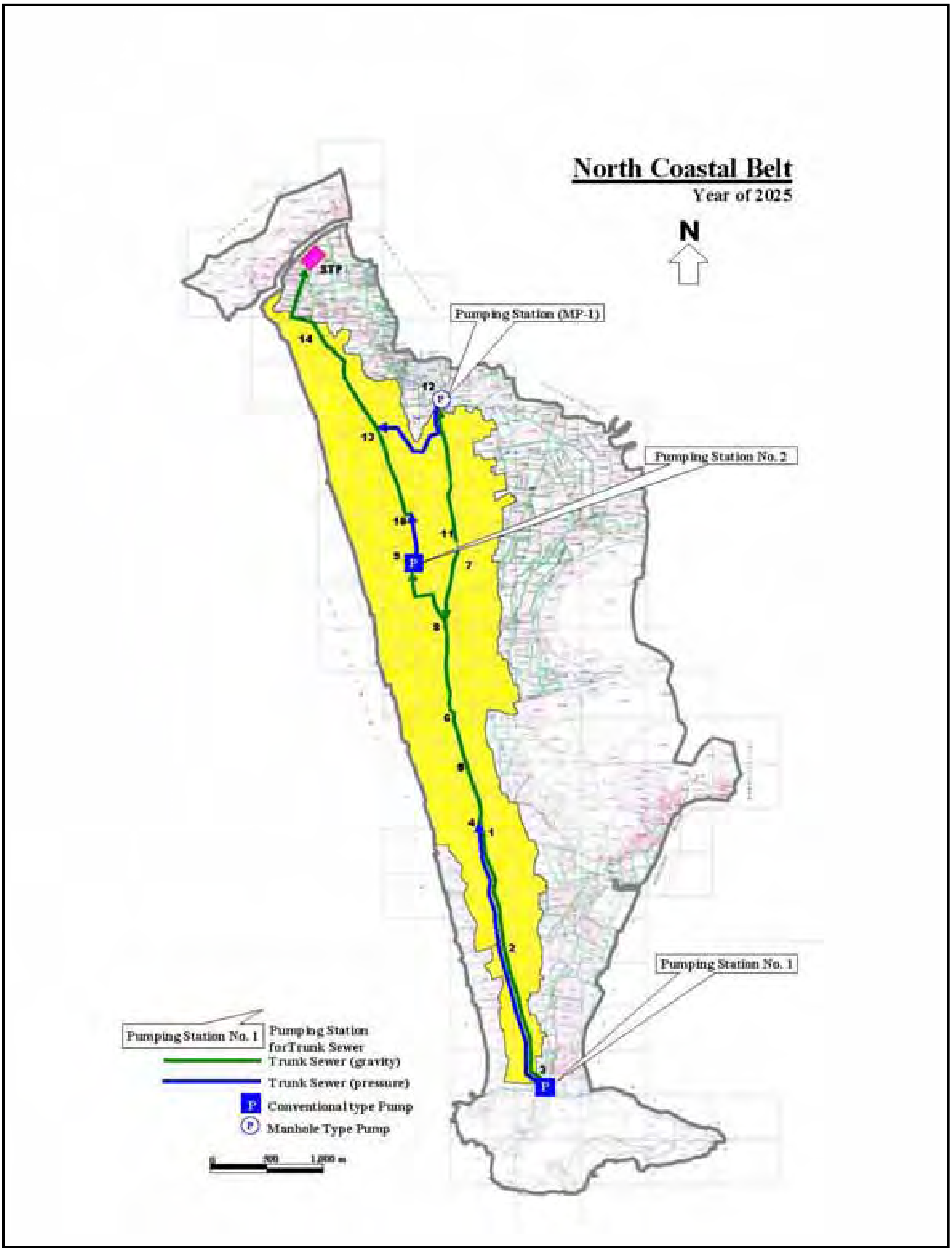


Figure 63.10 Proposed Sewerage Service Area of North Coastal Belt

(3) Pumping Station

1) Pumping Station No. 1

Pumping Station No.1 should be located at south edge of sewerage service area of Candolim Village. This pumping station should receive whole wastewater of Candolim Village. The collected wastewater should be conveyed via a 250 mm diameter pressure main about 2,500 m to outfall manhole near the boundary between Candolim and Calangute.

2) Pumping Station No. 2

Pumping Station No.2 should be located in lowland near the center of Calangute Village. This pumping station should receive about half of the wastewater of Calangute Village and whole wastewater of Candolim Village. Wastewater should be conveyed via a 400 mm diameter pressure main about 500 m to outfall manhole, which finally flows go to the STP by gravity.

Another one manhole type pumping stations are proposed for sewage of trunk sewer as shown in Figure 63.10. Design flows considered for this pumping station are presented in Table 63.45. The calculated capacities, requirements of pumps and pressure main for this pumping station are presented in Table 63.46.

Table 63.45 Design Flow of Pumping Stations for Trunk Sewer

Pumping Station	Location (Node No.)	Design Flow (peak) (lps)	Remarks
Pumping Station No. 1	3	87.7	
Pumping Station No. 2	9	215.4	
Pumping Station (MP-1)	12	46.9	Manhole type

Table 63.46 Design of Pumping Stations and Pressure Mains for Trunk Sewer

Pumping Station	Design Flow (peak) (lps)	Pump			Pressure Main	
		Capacity		Nos. of Pump	Diameter (mm)	Length (m)
		(lps)	(m^3/min)			
Pumping Station No. 1	87.7	44	2.6	2 + (1-standby)	250	2,500
Pumping Station No. 2	215.4	27 54 108	1.6 3.2 6.5	2 + (0-standby) 1 + (0-standby) 1 + (1-standby)	400	500
Pumping Station (MP-1)	46.9	47	2.8	1 + (1-standby)	200	1,200

In addition to pumping stations mentioned above, some manhole type small pumping stations should be implemented in this area.

(4) Sewage Treatment Plant (Baga STP)

The proposed STP site is a grassland for cattle grazing and does not require any resettlement. The ground level is lower than the surrounding road surface. The following strategies are used for developing layout plan of the proposed STP.

- The proposed facilities, which may release odor and/or noise will be located further away from the nearest residential area and surrounding roads.
- The ground level of the site will be raised to avoid flooding in rainy seasons.
- Landscaping will be considered for buffer zone, which surround the STP along the site boundaries in order to improve amenity in the surrounding area.

1) Sewage and Sludge Treatment Method

As discussed in Section 6.2, the proposed sewage treatment plant will use oxidation ditch process. It will also have sand filters as advanced treatment for water reuse, and a storage tank for filtered water. Sludge will be digested to reduce number of pathogen in sludge, which will make it safe for reuse.

2) Basic Conditions

The proposed STP facilities are designed based on the basic conditions as shown in Table 63.47.

Table 63.47 Basic Conditions for the Baga STP

Item	Basic Conditions	
Design Sewage Flow	m^3/day	11,200 m^3/day
Design Sewage Quality	BOD	In: 240 mg/l, Out: 30 mg/l
	SS	In: 200 mg/l, Out: 50 mg/l
Discharge Point	Baga River	
Required STP Area	15,800 m^2	
Sewage Treatment	Lift pump + Screen/Grit chamber + Oxidation Ditch + Final settling + Sand filter + Disinfection	
Sludge Treatment	Thickening + Digestion + Dewatering	

3) Sewage Treatment Facilities

The capacity calculation based on the basic conditions for the Baga STP is shown in Volume IV Appendix M63.3 Capacity Calculation of Sewerage Treatment Facilities, the summary of the main proposed facilities of STP is shown in Table 63.48.

Table 63.48 Summary of the Main Proposed Facilities for Baga STP

Facilities	Additional Facilities
Wet well	5mW×4mL×5mH×1basin
Pump equipment	$2.43m^3$/min×12kw×2units
	$4.86m^3$/min×23kw×1unit
	$9.72m^3$/min×46kw×2(1)unit
Grit chamber	1.8mW×3.6mL×0.3mH×2basins
Screen	Mechanical screen
Oxidation ditch	4mW×146mL×3mH×4basins
Final settling tank	16mdia×4mH×4basins
Sand filter	5mW×6mL×2basins
Disinfection tank	2mW×10mL×1mH×6passes
Thickening tank	5mdia×3mH×2basins
Digestion tank	6.5mdia×8mH×2basins
Gas tank	10.5mdia×5mH×1basins
Dewatering	Centrifugal×$5.0m^3$/hr×2(1)units

6.4 Required Land Area for New Sewage Treatment Plants

Among the eight (8) service blocks in the Study area, six (6) service blocks, except Panaji and Margao, requires new STP sites. Land requirement for those STPs, which use oxidation ditch process, was calculated taking area for maintenance roads and buffer zones into account. The detailed calculations are shown in Volume IV Appendix M64.1 Calculation of Required Land Area for STP. The results are summarized in Table 64.1.

Table 64.1 Required Land Area for New Sewage Treatment Plants

Sewage Treatment Plant	Required Land Area (m^2)
St. Cruz	4,000
Porvorim	11,000
Ponda	5,300
Mapusa	15,500
Colva (South coastal belt)	3,500
Baga (North coastal belt)	15,800

6.5 Summary of the Proposed Sewerage Facilities

The proposed sewerage facilities in the Study Area are summarized in Table 65.1.

Table 65.1 Summary of Proposed Facilities for STP (1/2)

Sewerage Area	Facility		Existing Facility	Additional Facility
Panaji and Surroundings	Sewer		Trunk Sewer : 150-700mm L=11,800m	Trunk Sewer : 200-500mm L=9,450m
			Branch Sewer : 150-300mm L=27,300m	Branch Sewer : 150mm L=55,700m
	Pumping Station		Number of conventional type : 8	Number of conventional type : 2
			Number of manhole type : 0	Number of manhole type : 6
	Sewage Treatment Plant	Wet well	15.0mdia.×5.0mH×1basin	-
		Pump equipment	150mm×4.35m3/min×16.0m×22kw×2units	200mm×5.20m3/min×16.0m×26kw×1unit
			200mm×9.77m3/min×16.0m×55kw×3(1)units	-
		Grit chamber	4.8mW×4.8mL×1basin	-
		Screen	Mechanical screen×1unit with 20mm bar opening	-
		SBR tank	22.0mW×40.0mL×4.0mH×2basins	24.0mW×47.0mL×4.0mH×2basins
		Sand filtration tank	-	5.0mW×5.5mL×4basins
		Disinfection tank	2.6mW×9.5mL×1.0mH×9passes	-
		Sludge thickening tank	-	6.0mdia.×3.0mH×2basins
		Sludge digestion tank	-	8.0mdia.×8.0mH×2basins
		Sludge dewatering	Centrifugal type×20.0m3/hr×2units	-
		Sludge drying bed	7.5mW×15.0mL×0.3mH×18basins	-
St. Cruz	Sewer		-	Trunk Sewer : 200-400mm L=2,950m
			-	Branch Sewer : 150mm L=14,900m
	Pumping Station		-	Number of conventional type : 1
			-	Number of manhole type : 3
	Sewage Treatment Plant	Wet well	-	3.5mW×2.0mL×4.0mH×1basin
		Pump equipment	-	100mm×1.30m3/min×15.0m×7kw×2units
			-	150mm×2.60m3/min×15.0m×13kw×2(1)units
		Grit chamber	-	0.6mW×2.9mL×0.3mH×2basins
		Screen	-	Mechanical screen×1unit with 20mm bar opening
		Oxidation ditch tank	-	4.0mW×67.0mL×3.0mH×2basins
		Final settling tank	-	10.5mdia.×4.0mH×2basins
		Sand filtration tank	-	3.0mW×2.5mL×2basins
		Disinfection tank	-	1.0mW×4.5mL×1.0mH×6passes
		Sludge thickening tank	-	3.0mdia.×3.0mH×2basins
		Sludge digestion tank	-	6.5mdia.×6.0mH×1basin
		Sludge dewatering	-	Centrifugal type×2.0m3/hr×2(1)units
Porvorim	Sewer		-	Trunk Sewer : 200-600mm L=11,650m
			-	Branch Sewer : 150mm L=67,200m
	Pumping Station		-	Number of conventional type : 2
			-	Number of manhole type : 9
	Sewage Treatment Plant	Wet well	-	4.2mW×4.0mL×4.0mH×1basin
		Pump equipment	-	125mm×1.65m3/min×15.0m×8kw×2units
			-	200mm×3.30m3/min×15.0m×16kw×1unit
			-	250mm×6.60m3/min×15.0m×31kw×2(1)units
		Grit chamber	-	1.3mW×3.5mL×0.3mH×2basins
		Screen	-	Mechanical screen×1unit with 20mm bar opening
		Oxidation ditch tank	-	4.0mW×100.0mL×3.0mH×4basins
		Final settling tank	-	13.0mdia.×4.0mH×4basins
		Sand filtration tank	-	5.0mW×4.0mL×2basins
		Disinfection tank	-	1.5mW×7.0mL×1.0mH×8passes
		Sludge thickening tank	-	5.0mdia.×3.0mH×2basins
		Sludge digestion tank	-	10.0mdia.×8.0mH×1basin
		Sludge dewatering	-	Centrifugal type×4.0m3/hr×2(1)units
Margao	Sewer		Trunk Sewer : 300-1200mm L=11,000m	Trunk Sewer : 200-700mm L=9,000m
			Branch Sewer : 150-350mm L=33,600m	Branch Sewer : 150mm L=50,800m
	Pumping Station		Number of conventional type : 0	Number of conventional type : 2
			Number of manhole type : 0	Number of manhole type : 1
	Sewage Treatment Plant	Wet well	12.3mdia.×15.0mH×1basin	-
		Pump equipment	- mm×3.33m3/min×12.0m×25HP×2units	350mm×12.68m3/min×12.0m×48kw×2(1)units
			- mm×6.66m3/min×12.0m×50HP×2units	-
		Grit chamber	1.25mW×8.0mL×0.35mH×1basin	1.3mW×9.5mL×0.35mH×1basin
		Screen	Mechanical screen×1unit with 20mm bar opening	-
		Primary settling tank	18.0mdia.×3.0mH×1basin	16.5mdia.×3.0mH×1basin
		Activated sludge tank	12.0mW×33.0mL×3.0mH×1basin	12.0mW×24.0mL×4.0mH×2basins
		Secondary settling tank	21.0mdia.×3.0mH×1basin	18.0mdia.×4.0mH×2basins
		Sand filtration tank	-	5.0mW×5.5mL×4basins
		Disinfection tank	-	2.0mW×10.5mL×1.2mH×9passes
		Sludge thickening tank	-	8.5mdia.×3.0mH×2basins
		Sludge digestion tank	18.0mdia.×10.65mH×1basin	-
		Sludge dewatering	-	Centrifugal type×9.0m3/hr×2(1)units
		Sludge drying bed	12.4mW×12.8mL×0.3mH×14basins	5.0mW×4.5mL×0.3mH×2basins

Note : The length of pressure main is included in the Trunk sewer

Table 65.1 Summary of Proposed Facilities for STP (2/2)

Sewerage Area	Facility		Existing Facility	Additional Facility
Ponda	Sewer		-	Trunk Sewer : 250-450mm L=3,400m
			-	Branch Sewer : 150mm L=24,100m
	Pumping Station		-	Number of conventional type : 1
			-	Number of manhole type : 10
	Sewage Treatment Plant	Wet well	-	4.0mW×2.5mL×4.0mH×1basin
		Pump equipment	-	150mm×1.82m3/min×15.0m×9kw×2units
			-	200mm×3.65m3/min×15.0m×18kw×2(1)units
		Grit chamber	-	0.7mW×3.5mL×0.3mH×2basins
		Screen	-	Mechanical screen×1unit with 20mm bar opening
		Oxidation ditch tank	-	4.0mW×92.0mL×3.0mH×2basins
		Final settling tank	-	12.5mdia.×4.0mH×2basins
		Sand filtration tank	-	3.0mW×3.0mL×2basins
		Disinfection tank	-	1.0mW×5.0mL×1.0mH×8passes
		Sludge thickening tank	-	3.5mdia.×3.0mH×2basins
		Sludge digestion tank	-	6.0mdia.×8.0mH×1basin
		Sludge dewatering	-	Centrifugal type×1.5m3/hr×2(1)units
Mapusa	Sewer		-	Trunk Sewer : 250-700mm L=8,700m
			-	Branch Sewer : 150mm L=47,000m
	Pumping Station		-	Number of conventional type : 1
			-	Number of manhole type : 2
	Sewage Treatment Plant	Wet well	-	4.5mW×4.0mL×5.0mH×1basin
		Pump equipment	-	150mm×2.11m3/min×15.0m×10kw×2units
			-	200mm×4.22m3/min×15.0m×20kw×1unit
			-	300mm×8.44m3/min×15.0m×40kw×2(1)units
		Grit chamber	-	1.6mW×3.6mL×0.3mH×2basins
		Screen	-	Mechanical screen×1unit with 20mm bar opening
		Oxidation ditch tank	-	4.0mW×142.0mL×3.0mH×4basins
		Final settling tank	-	15.5mdia.×4.0mH×4basins
		Sand filtration tank	-	5.0mW×5.5mL×2basins
		Disinfection tank	-	2.0mW×10.0mL×1.0mH×6passes
		Sludge thickening tank	-	6.0mdia.×3.0mH×2basins
		Sludge digestion tank	-	8.0mdia.×8.0mH×2basins
		Sludge dewatering	-	Centrifugal type×4.5m3/hr×2(1)units
Colva (South Coastal Belt)	Sewer		-	Trunk Sewer : 200-300mm L=3,650m
			-	Branch Sewer : 150mm L=13,200m
	Pumping Station		-	Number of conventional type : 1
			-	Number of manhole type : 2
	Sewage Treatment Plant	Wet well	-	3.5mW×1.9mL×3.5mH×1basin
		Pump equipment	-	100mm×1.15m3/min×15.0m×6kw×2units
			-	150mm×2.29m3/min×15.0m×11kw×2(1)units
		Grit chamber	-	0.6mW×2.6mL×0.3mH×2basins
		Screen	-	Mechanical screen×1unit with 20mm bar opening
		Oxidation ditch tank	-	4.0mW×59.0mL×3.0mH×2basins
		Final settling tank	-	10.0mdia.×4.0mH×2basins
		Sand filtration tank	-	4.0mW×2.0mL×2basins
		Disinfection tank	-	1.0mW×4.0mL×1.0mH×6passes
		Sludge thickening tank	-	3.0mdia.×3.0mH×1basins
		Sludge digestion tank	-	5.0mdia.×6.0mH×1basin
		Sludge dewatering	-	Centrifugal type×1.0m3/hr×2(1)units
Baga (North Coastal Belt)	Sewer		-	Trunk Sewer : 200-800mm L=14,100m
			-	Branch Sewer : 150mm L=71,200m
	Pumping Station		-	Number of conventional type : 2
			-	Number of manhole type : 1
		Wet well	-	5.0mW×4.0mL×5.0mH×1basin
		Pump equipment	-	150mm×2.43m3/min×15.0m×12kw×2units
			-	250mm×4.86m3/min×15.0m×23kw×1unit
			-	300mm×9.72m3/min×15.0m×46kw×2(1)units
		Grit chamber	-	1.8mW×3.6mL×0.3mH×2basins
		Screen	-	Mechanical screen×1unit with 20mm bar opening
		Oxidation ditch tank	-	4.0mW×146.0mL×3.0mH×4basins
		Final settling tank	-	16.0mdia.×4.0mH×4basins
		Sand filtration tank	-	5.0mW×6.0mL×2basins
		Disinfection tank	-	2.0mW×10.0mL×1.0mH×6passes
		Sludge thickening tank	-	5.0mdia.×3.0mH×2basins
		Sludge digestion tank	-	6.5mdia.×8.0mH×2basins
		Sludge dewatering	-	Centrifugal type×3.0m3/hr×2(1)units

Note : The length of pressure main is included in the Trunk sewer

6.6 Master Plan for On-site and Decentralized Facilities

6.6.1 General

On-site system is controlled by municipality/panthayat as same as solid waste, although there are no decentralized system in Goa now.

Either on-site treatment facility or decentralized treatment facility will be proposed for the areas that are not covered by the proposed sewerage system discussed in Chapter 6.1.2. These areas are shown in Table 66.1. Appropriate treatment process for each area is discussed below.

Table 66.1 Areas of Decentralized and On-site System in the Study Area

Decentralized	Onsite (Septic Tank)
Ribandar	Part of St. Cruz
Part of Merces	Part of Merces
Part of South Coastal Belt	Part of Porvorim
	Part of Mapusa
	Part of South Coastal Belt
	Part of North Coastal Belt

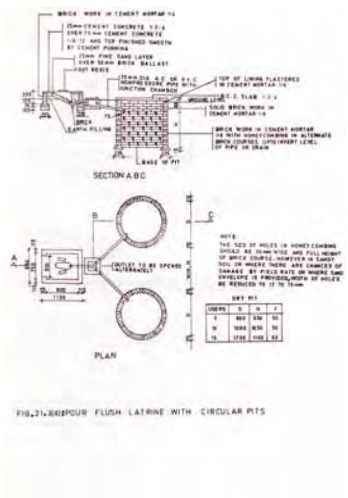
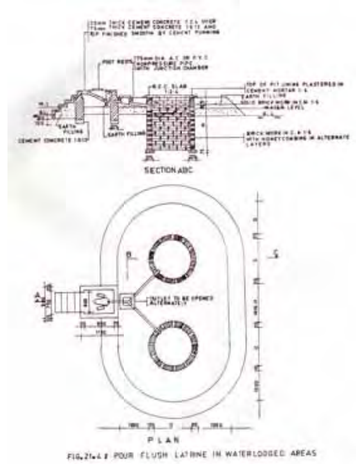
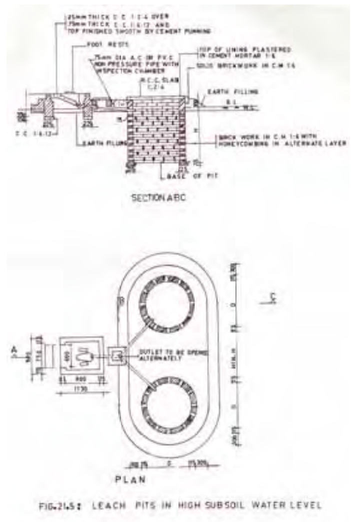
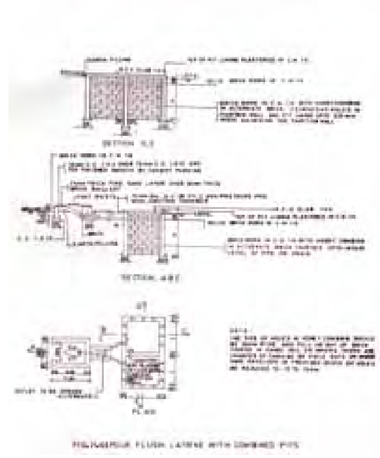
6.6.2 Onsite and Decentralized Facilities

The followings are recommended for onsite treatment facilities of residents, hotels, and factories outside the sewerage aiming to improve living environmental and to preserve water quality of public water body and groundwater.

(1) Promotion of Sulabh Latrine for Individual Houses

Sulabh Latrine, which has been developed in the mid 1940s is recommended for sanitation improvement as a better option than pit latrines and open defecation. Typical Sulabh Latrine, pour flush sealed latrine, is shown in Table 66.2, which is introduced by CPHEEO.

Table 66.2 Summary of Type of Pour Flush Water Seal Latrines

Item	Typical type	In water flooded Area	In high sub-soil water level area	In constrained space area
Sketch of Latrine				
Characteristic	The twin-pit design is introduced when one pit is full, the excretion is diverted to the second pit. The filled up pit can be conveniently emptied after 1.5 to 2 years, when most of the pathogens die off. The sludge can safety is used as manure. Thus the two pits can be used alternately and perpetually.	The pit top should be raised by 300 mm above the likely level of water above ground level at the time of water logging. Earth should then be filled well compacted all round the pits upto 1.0m distance from the pit and upto its top. The raising of the pit will necessitate rising of latrine floor also.	Where the sub-soil water level rises to less than 300 mm below ground level, the top of the pits should be raised by 300 mm above the likely sub-soil water level and earth should be filled all round the pits and latrine floor raised.	Where circular pits of standard sizes cannot be constructed due to space constraints, deeper pit with small diameter (not less than 750 mm), or combined oval, square or rectangular pits divided into two equal compartments by a partition wall may be provided. In case of combined pits, the partition wall should not have holes.

(2) Inspection and Monitoring on Onsite Treatment Facility

Through the observation of existing treatment facilities of large hotel and factories, proper operation was found as described in Chapter 3. Legislation set up for periodical monitoring is recommended to keep the wastewater treatment facilities operated properly.

(3) Improvement of Soakage Pit

In the coastal area, sewage overflows from septic tanks because of low ground percolation in soakage pits. One of the reasons of low percolation is clogging by suspend solids. A measure for preventing clogging is to build another soakage pit when the percolation becomes slow. The long-term infiltration rates of different soil types are presented in Table 66.3, taken from the CPHEEO manual. Periodical sludge removal from septic tanks is recommended to reduce suspended solids in overflow effluent.

Table 66.3 Long Term Infiltration Rates of Different Soil Types

Soil Type	Long Term Infiltrative Loading Rate (l/m^2 per day)
Sand	50
Sandy Loam, Loams	30
Porous silty loams, Porous silty clay loams	20
Compact silty loams, Compact silty clay loams, Clay	10

(4) Promotion of Raised Toilet

To prevent flooding of septic tanks, toilet facility installation on raised mound could be effective.

(5) Promotion of High Efficiency Treatment System

For medium/small hotels and/or scattered apartments where soil percolation rate is low, high efficiency treatment system such as shown in Figure 66.1 (sedimentation + anaerobic digestion which introduced by CPHEEO) or in Figure 66.2 (packaged system, popular in Japan) shall be considered.

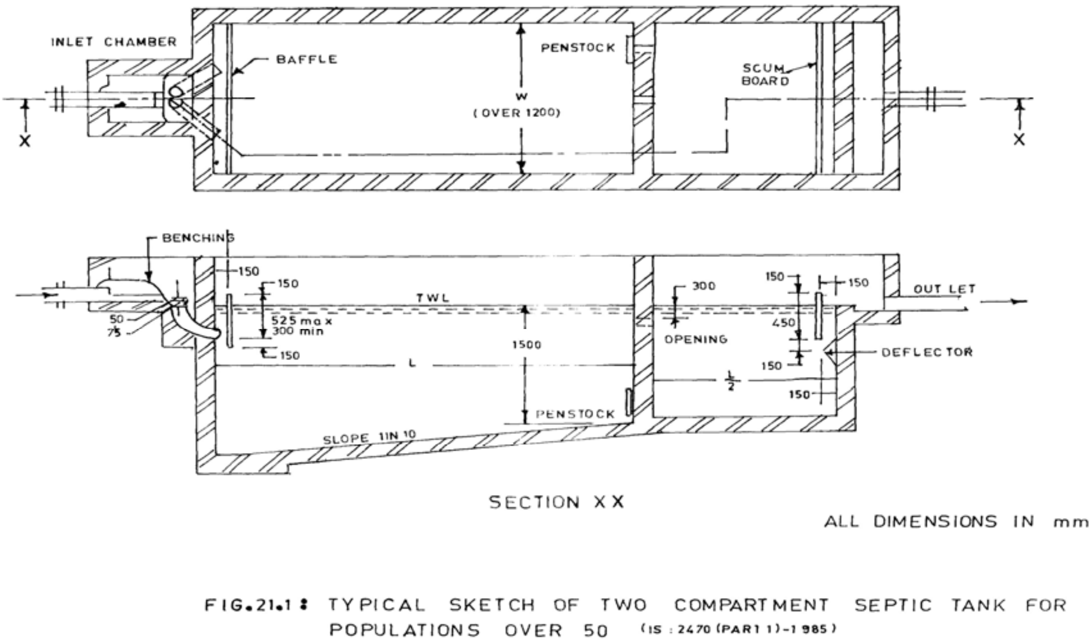


Figure 66.1 Onsite Treatment Facility introduced by CPHEEO

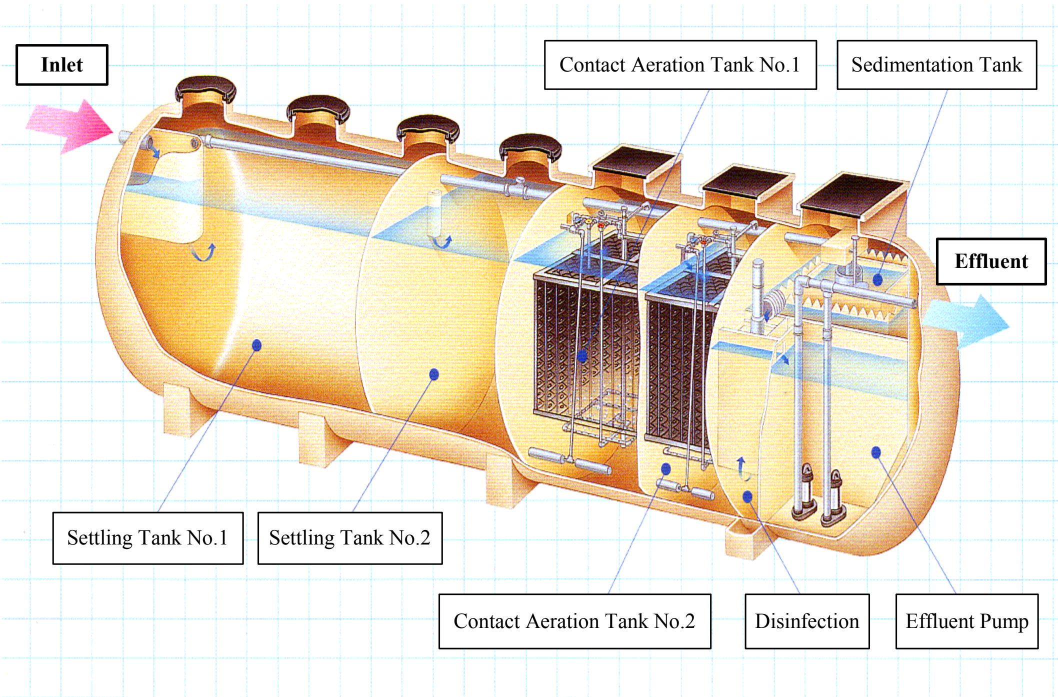


Figure 66.2 Typical Packaged Treatment System in Japan