CHAPTER 10
SEWERAGE SYSTEM PLANNING CONSIDERATIONS

10.1 GENERAL

The concept for the wastewater collection and final disposal is based on the fundamental engineering design criteria. The wastewater is to be collected through separate main, sub-main, branch and lateral sewers. Initially, wastewater treatment plant (WWTP) facilities will be minimal as appropriate but will be capable of expansion and upgrading, as dictated by the population increase or environmental quality requirements.

The concept proposed in this master plan suggests phasing the construction of sewers and WWTP facilities. The result will be an immediate benefit to a significant portion of the Master Plan (M/P) Area population, commensurate with economic limitations.

At the onset, this plan encourages the continued use and proper operation and maintenance of the existing sewerage facilities. Septic tanks or other inefficient sanitation facilities will be finally abandoned as the new sewer reticulations, main collector/interceptor sewers are expanded, and WWTP facilities with sufficient capacities are provided.

By the year 2015, it is assumed that the bulk of the Santiago City and its surrounding areas will have separate sewers collecting and conveying the wastewaters to the WWTPs, whereas in the sparsely populated districts, the wastewater would be treated with the private on-site sanitary systems.

10.2 WASTEWATER COLLECTION SYSTEM

10.2.1 SEWER PLANNING CONCEPTS

The concept for wastewater collection under this program is the immediate collection of untreated and/or partially treated (septic tank or other sanitary system effluents) wastewater from the populated areas where public sewer systems exist.

In certain areas, outside the Study Area, that can reasonably be expected to contribute to the sewerage system, the sewers will be sized to include these additional flows wherever considered necessary. Any additional tributary areas should not be permitted without proper planning.

10.2.2 MAIN AND INTERCEPTING SEWERS

(1) General

The wastewater is normally conveyed by gravity flow to the points for discharge. The major interceptor and collector sewers profiles are examined and determined so that the requirement of lift pumping stations is minimized to raise the wastewater.

(2) Rafey Sewerage District

The Rafey is the largest District covering mainly the central portion of the Santiago built-up urban areas. Since the topography of the District is such that the ground surface generally declines from east to west, the main and intercepting sewers follow the major drainage basin pattern.

Because at the north of Arroyo Nibaje is a hilly land that prevents further wastewater flow by gravity toward the Rafey WWTP, the 1970 sewerage master plan envisaged a lift station in the area. This plan was later on altered and an inverted siphon was constructed to cross the Yaque del Norte River, so as to flow the wastewater continuously by gravity.

The inverted siphon consists of two separated sewer barrels of 24” and 30” in diameter. The sewers following the inverted siphon, laid near to the ground with one-meter earth covering, are connected to the main sewer line of 70” in diameter to relieve excess wastewater by gravity to the River during wet weather and/or power outage. A schematic layout of these sewers is shown in Figure 10.1.

The wastewater coming from the area at the upstream of the Arroyo Nibaje, presently being transported to the existing Embrujo WWTP, is to be diverted to the Rafey District for the following reasons:

- The trunk sewer serving the District runs along the Arroyo Hoya Del Caimito. Despite the fact that the upstream trunk sewer size is 24” in diameter, the downstream connecting trunk sewer from Autopista Duarute and afterward, the size is 12” in diameter. Hence, the downstream trunk sewer capacity is insufficient to handle all the incoming wastewater from the upstream trunk sewer; and
- The tributary area of the District is relatively wide compared with other Districts, and large-scale housing development programs have been intensively ongoing. Consequently, the treatment capacity of the existing WWTP will be overloaded and not capable of treating the inflowing wastewater.

Construction of a portion of the trunk sewer “Corector 10 Hasta Pontezuela,” transporting the wastewater to the Rafey WWTP, was once implemented, but interrupted later on and left uncompleted. The layout of the existing and planned main sewers is illustrated in Figure 10.2.

(3) Embrujo Sewerage District

Since the existing Imhoff tank WWTP in Urb. Thomen can hardly produce the effluent quality complying with the wastewater discharge quality standards, the wastewater should be diverted to the Embrujo WWTP in the future (For the layout, see Figure 10.2). For this purpose, a new main sewer is to be laid along the existing drainage channel. The new sewers will also collect the wastewater from the existing sewers in the low-lying areas along the drain.

(4) Cienfuegos and Los Salados Sewerage Districts

Cienfuegos and Los Salados Sewerage Districts serve the tributary of the Rio Jacagua, and hence the treated wastewaters are discharged to the downstream of the Yaque Del Norte River, at a point distant away from the central urban districts. In the Cienfuegos District, sewers are well provided. However, as the flow capacities of the originally laid sewers have become no longer sufficient to handle the ever-increasing wastewater inflows, a new parallel sewer was laid along the Av. Circunvalacion.

In the Los Salados District the sewer networks have been almost completed. A plan has now been underway to construct a trunk sewer line along the Arroyo Salado River and its tributaries. Upon completion of this trunk sewer the wastewater from some of the existing sewered areas can be connected to the WWTP, which otherwise have no access to the public sewers.

These wastewater collection systems are schematically shown in Figure 10.3.

(5) Zona Sur and Herradura Sewerage Districts

Only portions of these Districts are sewered, and no WWTP exists. The Zona Sur District collects the wastewater mainly from the tributary of the Arroyo Hondo River, a branch of the Yaque del Norte River. Although this District adjoins the Rafey District, an independent WWTP is planned for the following reasons:

- To transport the wastewaters of this District to the Rafey WWTP, a large-scale pumping station will be required;

- The wastewater can be collected by gravity when the sewer is laid along the Arroyo Hondo; and
- The existing main sewer has not sufficient hydraulic capacity to send the wastewater to the Rafey WWTP, and it is evidently not economical and technically unfeasible to increase the existing main sewer capacity to handle the increased wastewater inflows.

A part of the Herradura District is already connected to the Rafey District, covering the Corona Plaza and Otra Band areas. The Corona Plaza area could be drained by gravity to the main sewers, while the sewers in the Otra Banda area is currently being drained to the Rafey District through the Otra Banda pumping station. All the wastewaters in this District could be transported to the new WWTP by gravity. However, as the new WWTP would be available in the later stage, the wastewater should be sent to the Rafey District until the new WWTP is completed.

(6) Tamboril Sewerage District

Most of the urban districts in the City is already served by the sewerage system. All the wastewater collected through the sewer reticulations is transported by gravity to the conventional activated sludge WWTP. In Tamboril City, as a satellite town of the Santiago Metropolitan Area, housing construction works are intensively on going. The housing developments are underway mainly at the hilly areas north of the City center, where the sewerage system needs to be provided in the near future.

Generally, the sewered area has steep ground surface slopes that eliminate the need of lift stations, and all the wastewater can flow by gravity all the way toward the WWTP.

The outline of wastewater collection system layout is shown in Figure 10.4.

(7) Licey Sewerage District

The Licey City situates at the right bank of the Rio Licey, and the urban district has been expanding in the areas at both sides of the Cabretera Duarte. As the new international airport is now under construction at the southern part of the City, access roads construction is also planned. Under the circumstance, it is expected that City has a great potentiality to further develop in the near future. In the Uveralarea located close to the Airport, intensive housing development programs are taking place, and the City is quite likely to expand toward this area before long. The new WWTP site is selected in the Uveral area for the following reasons:

- All the wastewater could be transported to the WWTP by gravity;
- Sufficiently wide land area could be secured for the WWTP;
- An appropriate drainage is located close to the site to dispose of the effluent;
- The site is rather isolated and will have no significant impacts to the surrounding area; and
- There seems to be not much constraint for procurement of the land.

The topography of the City is that the Rio Licey runs from the north to south accordingly the ground surface declines toward the south. As such, the sewers can be laid following the natural topography. The proposed outline of wastewater collection system is illustrated in Figure 10.5.

10.2.3 SEWER DESIGN BASIS

The principal items of the wastewater collection system design criteria are discussed here. In general, and except for special reasons, the sewer systems are planned and designed on the basis of the following criteria:

(1) Design Period

In general, sewers should be designed for the estimated tributary population in the year 2015, except for part of the system that can be readily increased in capacity.

(2) Design Factors

In determining the required capacities of sanitary sewers the following factors are to be considered:

- Maximum hourly wastewater flow; The maximum daily flow rate x 1.5 + infiltration + industrial wastewater;
- Additional maximum wastewater flows from any facility that is justified necessary;
- Infiltration of 25 liters per capita daily (lpcd);
- Topography of the area, watersheds, ground surface slopes, etc.;
- Depth of excavation for sewers, in general less than 6 m; and
- Pumping requirements.

(3) Design Bases

Public sewers are to be not less than 200 mm in diameter except for house connection pipes. The Meaning's equation should be used in principle for gravity sewers.

(4) Velocity of Flow

All sewers should be designed and constructed to give mean velocities, when flowing 60 percent depth, of not less than 0.6 m/sec, based on the Meaning's formula. The velocity shall not exceed 3 m/second in any type of sewers to protect sewer erosion. The sewer slopes shall be such that flow the wastewater with mean velocities of more than 0.6 m/sec when flowing full.

(5) Alignments

Sewers shall be laid in general with straight alignment between manholes.

(6) Increasing Size of Sewers

When a smaller sewer joins a larger one, the invert of the larger sewer should be lowered sufficiently to maintain the same energy gradient. An approximate method for securing these results is to place the sewer crown of both sewers at the same elevation.

(7) Joints and Infiltration

Sewer joints shall be so designed as to minimize infiltration and to prevent the entrance of roots or other obstacles.

(8) Manholes

Manholes shall be installed at the end of each line; at all changes in grade, size, or alignment; at all intersections; and at distances as shown in the following:

Maximum Manhole Spacing

Sewer Diameters (mm)	Maximum Manhole Spacings (m)
200 or smaller	30
200 to 500	45
600 to 1,000	80
1,000 or larger	100

10.2.4 DESIGN OF PUMPING STATIONS

Although sewers are designed in principle to flow the wastewater by gravity, there may be some locations where aid of lift pumping stations can be economically justifiable. In such cases, the wastewater pumping stations may be designed. All pumping equipment, piping and conduits shall be designed to carry the expected peak flow rates.

For a large pumping station to lift the wastewater sub-main or main sewers should generally be of a dry well type. Provision shall be made to facilities removing pumps and motors. Suitable and safe means of access shall be provided to dry wells of pumping stations and shall be provided to wet wells containing either bar screens or mechanical equipment requiring inspection or maintenance.

For intermediate wastewater pumping stations require for lifting the wastewater of branch and/or sub-main sewers should be of submersible type provided in a manhole or similar structures. Submersible pumps shall be readily removable and replaceable without dewatering the wet wells and with continuity of operation of the other unit or units.

10.3 WASTEWATER TREATMENT SYSTEM

10.3.1 TARGET EFFLUENT QUALITIES

The designed WWTPs' effluent qualities to the public waters are determined based the discharge wastewater quality standards to surface or ground serve as a basis for the WWTPs assessment.

These requirements involve certain degree of removal of BOD_5 and other nutrient items as shown below:

- BOD_5 = 35 mg/L
- SS = 35 mg/L
- T-N = 18 mg/L
- T-P = 2 mg/L
- Total Coliform Number (MPN) = 1,000/100mL

10.3.2 ALTERNATIVE TREATMENT PROCESSES FOR NEW WWTPS

(1) Alternative Processes

The minimum requirements for a selected treatment process are to achieve the removal efficiencies that meet the effluent quality standards. Any treatment process with BOD5 and SS removal efficiencies of 80 percent or lower could hardly meet such requirements, since the raw wastewater BOD_5 concentrations in 2015 would be in the range of 160 to 200 mg/L or even higher in some sewerage districts.

In selecting appropriate treatment processes, those that could not comply with the quality requirements were first screened out from further study, and the following two processes have been assessed:

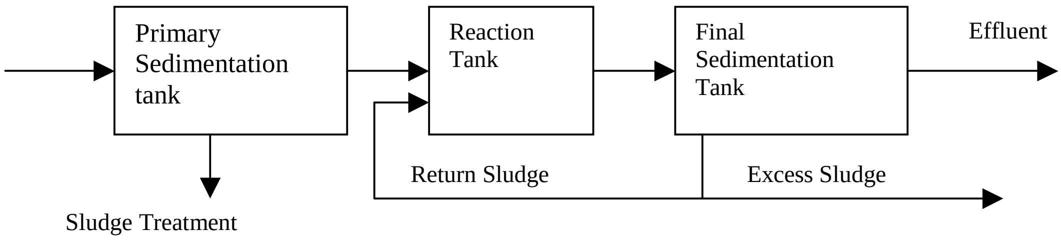
- Conventional activated sludge.
- Oxidation ditch.

For the comparison of the above alternative processes such important features as capital and O/M costs, land requirements, complexity of process O/M, sludge production, organics removal efficiency, etc. have been evaluated.

(2) Conventional Activated Sludge Process

The process comprises grit chamber, primary sedimentation tank, aeration tank, final sedimentation tank, chlorine contact tank, sludge thickeners, sludge digester, sludge dewatering facility, and other auxiliary facilities. The wastewater is commonly aerated for a period of 6 to 8 hours based on the average design flow in the presence of a portion of the secondary sludge. The rate of sludge return expressed as a percentage of the average wastewater design flow is normally about 25 percent, with minimum and maximum rates of 15 to 75 percent.

The expected BOD removal efficiency of the conventional activated sludge process is 90 percent or higher when the system is properly operated. A typical flow sheet of a conventional activated sludge plant is illustrated in the following:

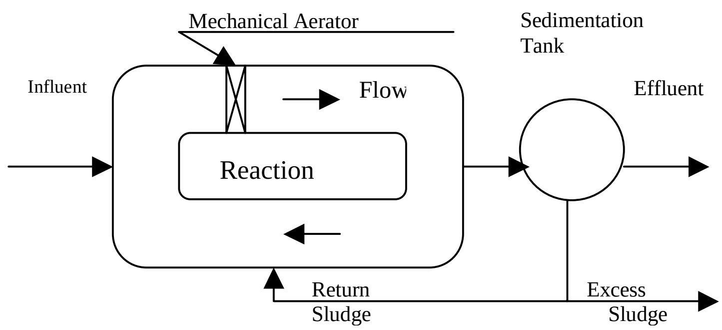


There are differences bases for planning the conventional activated sludge process depending on localities and surrounding conditions, such as sludge retention time in reaction tanks, densities of MLSS, depth of tanks, shape of tanks, etc.

(3) Oxidation Ditch Treatment Process

The oxidation ditch process is an extended aeration consisting of a ring-shaped channel about 1.0 to 3.0 meters deep and other facilities same as those for the extended aeration process. Aerators are placed across the ditch to provide aeration and circulation of the wastewater. The BOD removal efficiency is almost same as that of the conventional process.

The oxidation ditch t process generally omits primary sedimentation, and uses endless channels as reactor basins, which are provided with mechanical aeration equipment for aeration. Oxidation ditches are used to treat the wastewater by low-load activated sludge, and solids-liquid separation will be made in secondary settling tanks. A schematic flow sheet of the process is illustrated below:



Mechanical aeration equipment supplies the air to biological process, for mixing of the wastewater and activated sludge in ditches, recirculation of mixed liquor, and prevention of sludge settling. These are further discussed in more details in Appendix 9.2.1 "Selection of Wastewater Treatment Process."

10.3.3 SELECTION OF TREATMENT PROCESS

(1) General

Three new WWTPs at Zona Sur, La Herradura and Licey will be constructed by 2015, receiving the influent of 14,000 m^3/day, 11,800 m^3/day and 3,200 m^3/day, respectively. In selecting the most desirable treatment process, three typical treatment plant modules with treatment capacities of 15,000 m^3/day, 10,000m^3/day and 5,000m^3/day have been assessed for both the conventional activated sludge (CAS) and oxidation ditch (OD) processes as to the following fundamental features:

- Land requirements
- Performance and operational characteristics
- Capital costs
- Operation and maintenance costs

(2) Evaluation Results

The results of the above evaluation are summarized below:

Evaluation Results of Alternative Treatment Processes

Evaluation Items	Treatment Processes	Treatment Plant Capacities		
		15,000 m^3/d	10,000 m^3/d	5,000m^3/d
1. Land Requirements (ha.)	CAS	2.65	2.05	1.32
	OD	3.54	2.64	1.60
2. Capital Costs RD$1,000)	CAS	338,200	252,700	153,500
	OD	267,000	183,600	96,800
3. O/M Costs (RD$1,000/year)	CAS	21,600	16,946	11,200
	OD	12,900	10,169	6,728

Note: While the above values are sufficiently accurate for the comparison purpose, these should not be used for the detailed financial or engineering analyses

Land Requirements: The land area requirements for the OD and CAS methods with the same capacity is approximately 1.5 to 1 because of the shallower depth in the oxidation ditches and wider surface areas of sedimentation tanks. The land costs of the candidate plant sites in Santiago and Licey are however relatively low (about 200 to 300 RD$/$m^2$) and sufficiently wide areas are available for the plant facilities. There are neither residences nor commercial facilities located within 300 m from the boundaries of both sites, hence, will involve less socioeconomic and environmental problems. The wider land area requirements for the OD process does not significantly affect to the acquisition costs too. The land areas comprise all such necessary spaces as buildings, parking, roads, buffer zone, etc. The land requirements are calculated by the equations peculiar in the region.

Capital Costs: Capital costs for the WWTPs have been developed taking 2001 price level in the Santiago area into account. The costs of civil works, electrical and mechanical equipment, utilities and other facilities have been estimated and the functions developed for each process. The capital costs of the CAS process are generally 120 to 150 percent of OD process. As the OD process cost functions imply, the larger the OD plant capacity becomes, the less advantageous of the OD process will be. (For details refer to Volume III; Appendix-9.2.1 “Selection of Wastewater Treatment Process). As the OD process requires the wide surface area almost in proportion to the treatment capacity, the economy of scale cannot be expected.

O/M Costs: The OD method has much lower complexity in O/M of the system, whereas CAS method is rather complex. The overall higher O/M costs for CAS process may be explained that the process itself is rather complicated generally with anaerobic sludge digestion and dewatering facilities, more sensitive control required for aeration tank operation, primary sedimentation tanks, and chemical addition for sludge handlings. Compared to other treatment technologies, the overall energy requirements of OD process are low, operator attention is minimal, and chemical addition is not usually required. The cost estimates for the two processes indicate that OD process costs are about 70 percent of CAS process.

Performance and Operational Characteristics: The long hydraulic retention time and complete mixing in OD process minimizes the impact of a shock load or hydraulic surge. Properly operated OD process can achieve BOD, SS, and ammonia nitrogen removal of greater than 90 percent when required. The effluent SS concentrations are relatively high in OD process compared to CAS method.

OD process produces less sludge owing to extended biological activity during the activated sludge process. For small treatment plants, most of types of sludge processing are too complicated and require a high level of experience than is usually available, and for these reasons, sludge treatment facilities (both in CAS and OD methods) are generally not included in small-scale treatment plants.

For the small capacity treatment systems, particularly of OD process, the sludge treatment and disposal will not be of a big problem since the quantities are small and easy to handle, and therefore, can be removed. Sludge digestion may not be needed in oxidation ditch, but when it becomes really necessary to digest the excess sludge for the reason of disposal or another, sludge treatment process can be easily added to the plant.

Comparison of Alternative Treatment Processes

Treatment Process	Characteristics	BOD Removal
Conventional	High organic removal efficiency.	90% or higher
Activated Sludge	High sludge production expected.	
	Widely used process with long operation experience.	
	Complex O/M and require operators' skill.	
	Complex sludge treatment process required.	
	Sensitive against shock loadings.	
	Expandability, fair to good if designed conservatively.	
	Required treatment area is smaller than OD process. Consumption of energy is not quite high.	
	Difficult sludge drying with simple sand drying beds	
	More excess sludge production.	
	Aeration may be disturbed due to sand accumulation	
Oxidation Ditch	Relatively wide surface area required.	90% or higher
	Robust to shock loads and inflow fluctuations.	
	Applied for smaller scale plants than activated sludge.	
	Low sludge generation, easy dewatering on sand bed.	
	Relatively simple O/M, requires some skill of operators	
	Rather simple sludge treatment process required.	
	Nitrogen removal possible but poor bacterial removal.	
	More flexibility of system upgrade and expansion.	
	Primary clarifier may be omitted.	
	Moderate odor generation.	
	High sludge carryover.	
	Comparatively large ground area required.	

CAS process isn't resistant to shock organic or toxic loadings, while the OD process is in general resistant to such loadings. When process upgrade becomes mandatory in the future, the OD process can easily modify its operation method particularly for removing such nutrients of N and P. The above discussions are tabulated in the table above.

(3) Conclusions

The above analyses and discussions have led to the conclusion that both CAS and OD processes are considered suitable for the Santiago and Licey WWTPs. However, for the small-scale WWTPs like the new WWTPs, the OD process could be economically applied, because of easy O/M and robust process operation; high cost effectiveness; and more flexibility of future upgrading for further nutrient removal.

For these reasons, Zona Sur, La Herradura and Licey WWTPs should be designed based on the oxidation ditch process. A typical layout plan of 10,000m^3/day treatment capacity OD WWTP (Two trains of 5,000 m^3/day facilities) is shown in Figure 10.6.

10.3.4 SITE SELECTION OF WWTPS

The new Zone Sur and La Herradura and Licey OD WWTPs facilities will require flat lands of 3.6 ha, 3.0 ha, 1.2 ha respectively. Because of the existing sewer network layout, and topographic reasons, there are less alternative candidate sites for the WWTPs.

After a preliminary review the three most appropriate WWTP sites were first selected and further evaluated for their appropriateness as part of the integrated regional sewerage system plan *(Locations of the WWTPs sites see Figure 9.1 and Figure 9.3, Chapter 9)*.

The candidate sites for the Santiago Zona Sur, La Herradura and Licey WWTPs, sufficiently wide lands are available for acquisition. All sites are presently vacant, located at lower land areas that can receive the wastewater mostly by gravity. The Zona Sur WWTP site, though located at rather low-lying area, the ground elevations are sufficiently higher than river high water surface elevation. Thus, the treatment facilities will remain fully operational and accessible all the time.

Around the WWTP candidate sites are such that no major houses or buildings exist within 300 meters. Because of the circumstances and that there would be less nuisance to the surrounding areas such as odor, noise, vibrations, etc. caused by the construction and operation of the WWTPs. The access to the sites is available through public roads sufficiently wide for transporting heavy machines and materials for the construction and O/M of the WWTPs

10.3.5 PLANNING BASES FOR NEW WWTPS

Since many detailed guidelines are available for planning and design of WWTPs, the following bases are intended to indicate general planning bases for the preliminary planning and design of the WWTP facilities. More details of the preliminary engineering design of the OD process WWTPs are discussed in Volume III Supporting Report, Appendix-9.2.2 “Hydraulic and Organic Design of Component Facilities,” and Appendix 9.2.3 “Planning Concepts of Component Facilities”.

(1) Arrangement of Units

Component parts of the plant should be arranged for greatest operating convenience, flexibility, economy, and so as to facilitate installation of future units.

(2) By-Passes

Except where duplicate units are available, properly located and arranged by-pass structures shall be provided so that each unit of the plant can be removed from service independently. Where the discharge of raw wastewater is not permitted to the ground, a provision shall be considered to provide a storage facility of appropriate capacity.

(3) Emergency Power Failure

A standby power sources shall be provided to ensure the continuous operation of such important equipment as influent pumps, minimum number of aerators, and emergency lighting.

(4) Essential Facilities

Necessary facilities for operation and maintenance of the plant shall be provided, including:

- Water supply facilities;
- Drainage facilities;
- Plant roads and parking facilities;
- Service facilities; and
- Connecting conduits.

(5) Oxidation Ditches

The dimensions of each independent aeration ditch shall be such as to maintain effective mixing and utilization of air. Liquid depth should be in general 1.5 to 3.0 m. Inlets and outlets for each aeration tank unit shall be suitably equipped with valves, gates, stop gates, or other devices to permit controlling the flow to any unit and to maintain reasonably constant liquid level.

The mechanism and drive unit shall be designed for the expected conditions in the ditch in terms of the proven performance of the equipment. Multiple mechanical aeration unit installation shall be so designed as to meet the maximum air demand with the largest unit out of service. The design should also provide for varying amount of oxygen transferred in proportion to the load demand on the plant.

(6) Sedimentation Tank

Inlets should be designed to dissipate the inlets velocity, to distribute the flow equally and to prevent short-circuiting. Channels should be designed to maintain a velocity of at least 0.3 m/sec at one-half design flow. Provision shall be made for elimination or removal of floating materials in inlet structures having submerged ports.

Effective scum collection and removal facilities, including baffling, shall be provided ahead of the outlet weirs on all sedimentation basins. Provisions may be made for discharging of scum with the sludge.

Sludge well should be provided or appropriate equipment installed for reviewing and sampling the sludge. Provisions should be made to permit continuous sludge removal from final sedimentation basin when the sludge is returned to the ditches.

(7) Chlorine Contact Tank

Disinfection is accomplished with liquid chlorine or sodium hypochlorite. The chemical should be selected after due consideration of waste flow rates, application and demand rates, pH of wastes, cost of equipment and the chemical, availability and maintenance problems.

Required chlorinator capacity should be determined to have a sufficient contact time between the chlorine compound and the wastewater, but in general the contact time should be at least 15 minutes or longer to ensure bacterial destruction.

(8) Sludge Handling

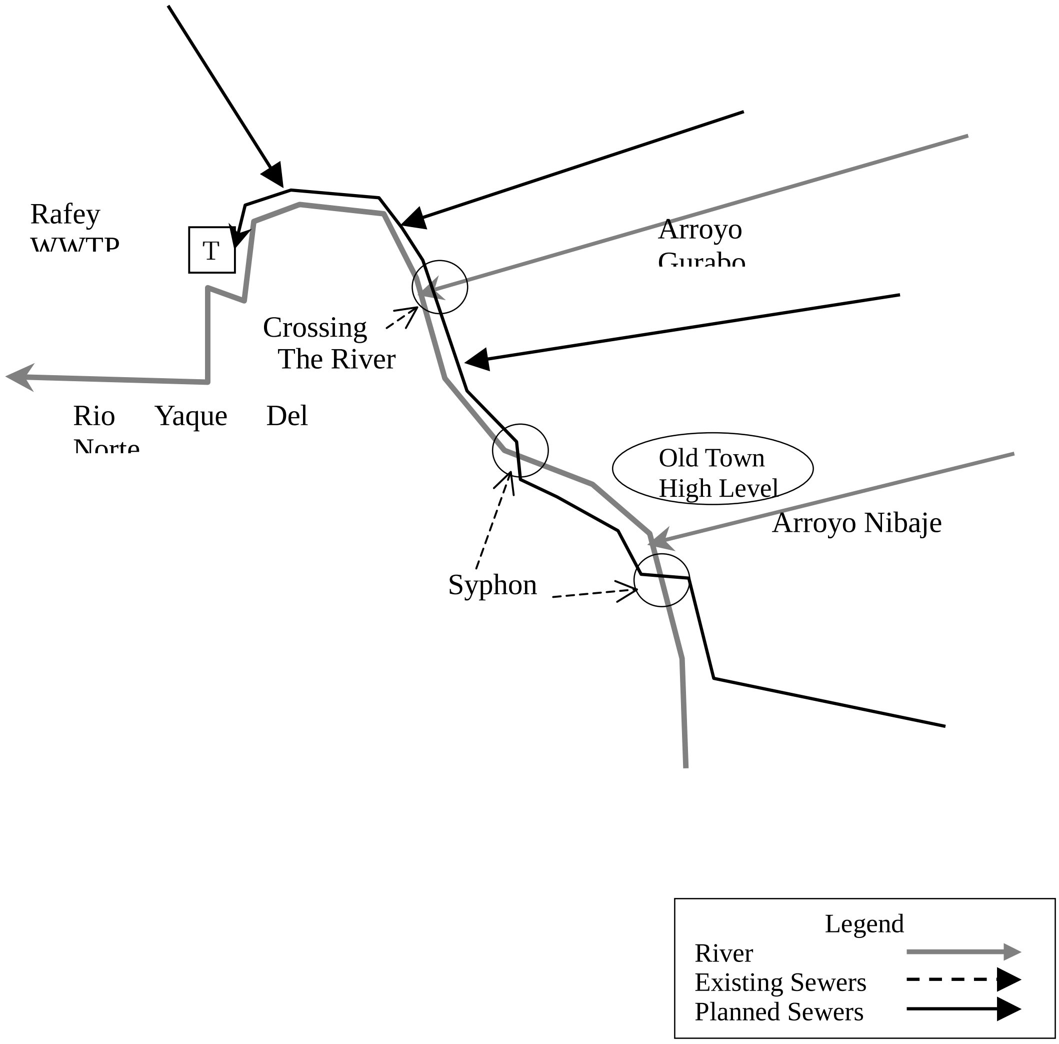
The excess sludge will be withdrawn from the final sedimentation tanks and transmitted to the sand drying beds. The dried sludge cake will be removed manually and disposed of to the municipal solids disposal sites.

10.4. INDUSTRIAL WASTEWATER MANAGEMENT

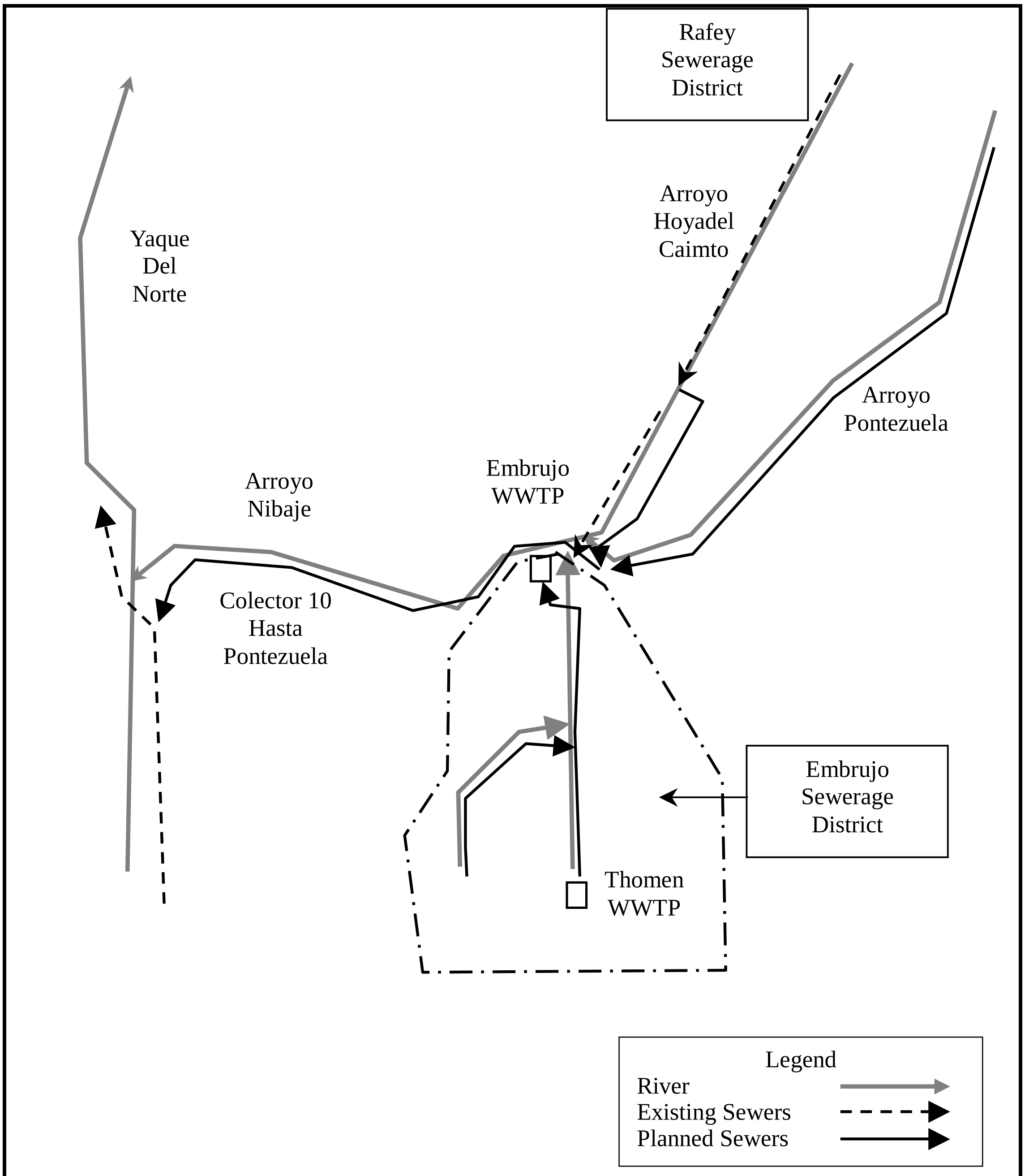
For the effective implementation and proper management of industrial wastewater, the industrial wastewater in the sewered area should in principle be led to the public sewers after appropriate pre-treatments, for the integrated treatment in the public-owned WWTPs. Before the discharge into sewerage or rivers, industrial wastewater should be properly treated in compliance with the respective effluent standards.

Basically, factories have to prepare the funds for providing wastewater treatment facilities by themselves. This will be a heavy burden on the industries particularly small-scale industries, and may result in the delay in implementation. In view of these, certain government financial assistance programs appropriate for local conditions should be established to promote the provision of industrial wastewater treatment systems. It is also suggested that some appropriate taxation arrangements be considered for national and/or local tax.

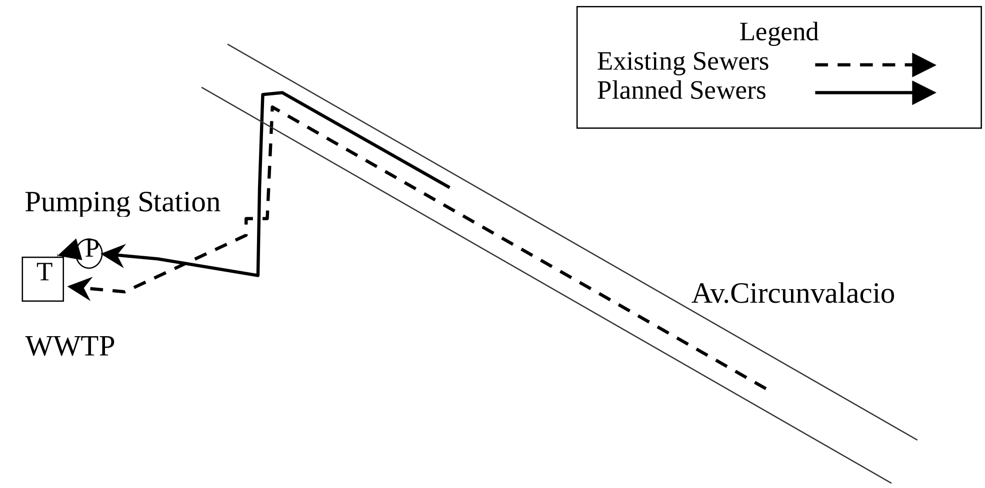
To ensure the governmental authorization and monitoring necessary for industrial wastewater management, both SEMARENA and CORAASAN may consider to organize a special unit. Further discussions on the industrial wastewater treatment and management are discussed in Appendix-10 “Industrial Wastewater Treatment and Management.”

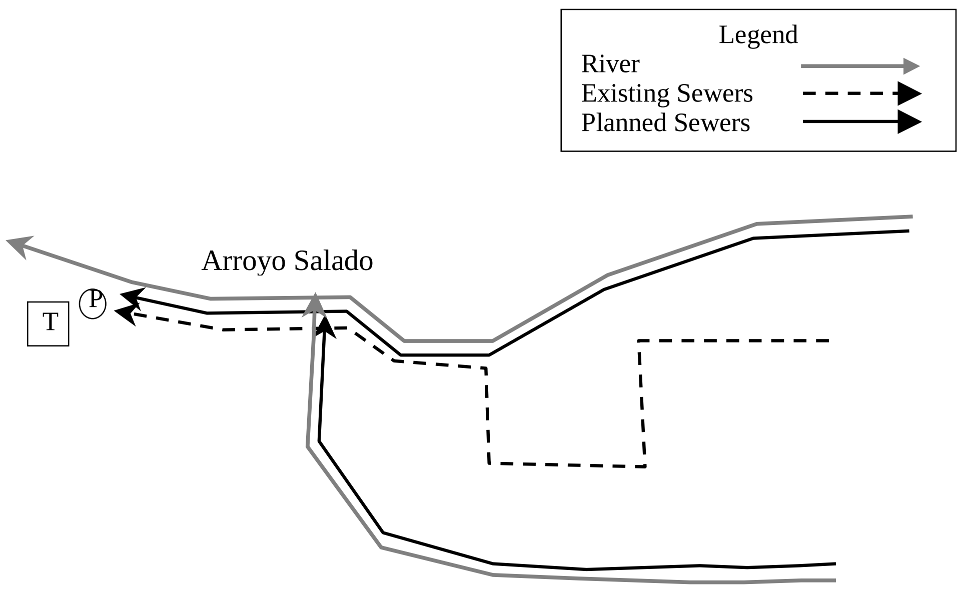


THE STUDY ON THE IMPROVEMENT OF SEWERAGE SYSTEM AND ENVIRONMENT IN THE CITY OF SANTIAGO	Figure 10.1 Outline of Rafey Wastewater Collection System
JAPAN INTERNATIONAL COOPERATION AGENCY	



THE STUDY ON THE IMPROVEMENT OF SEWERAGE SYSTEM AND ENVIRONMENT IN THE CITY OF SANTIAGO	Figure 10.2 Outline of Rafey and Embrujo Wastewater Collection System
JAPAN INTERNATIONAL COOPERATION AGENCY	

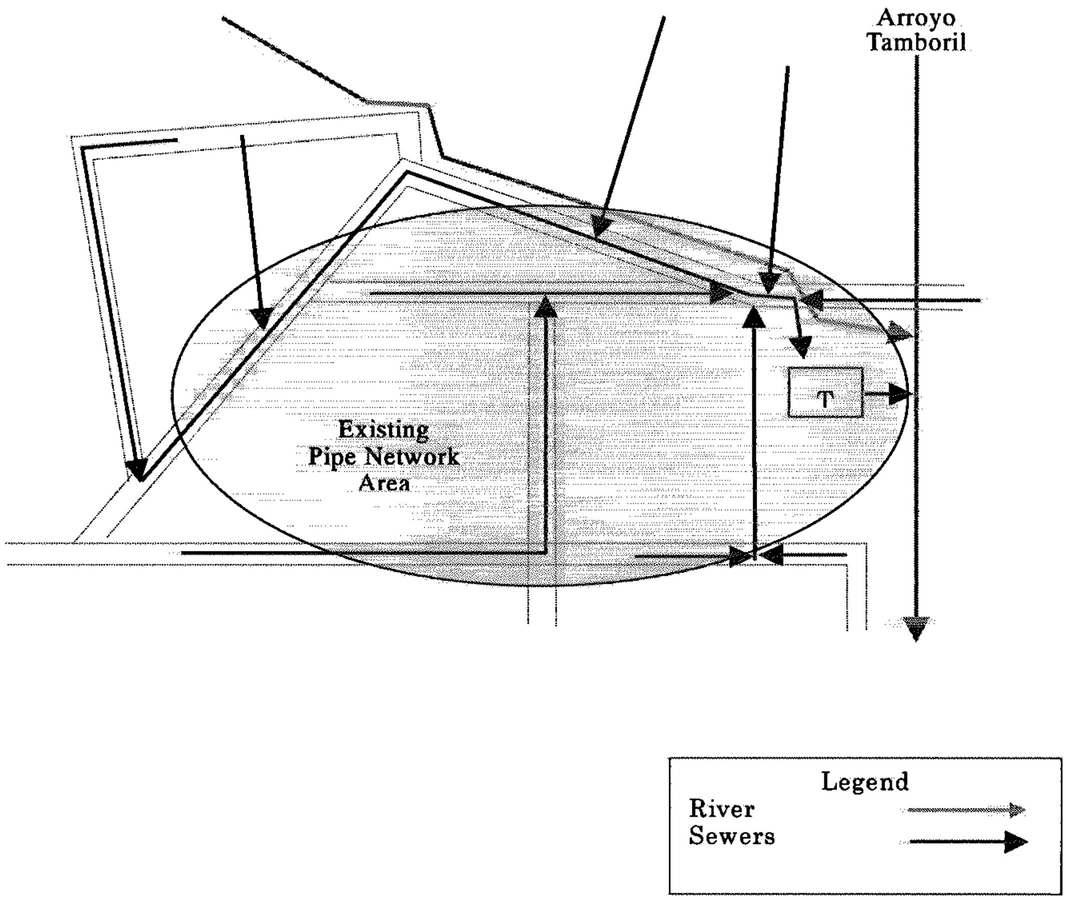




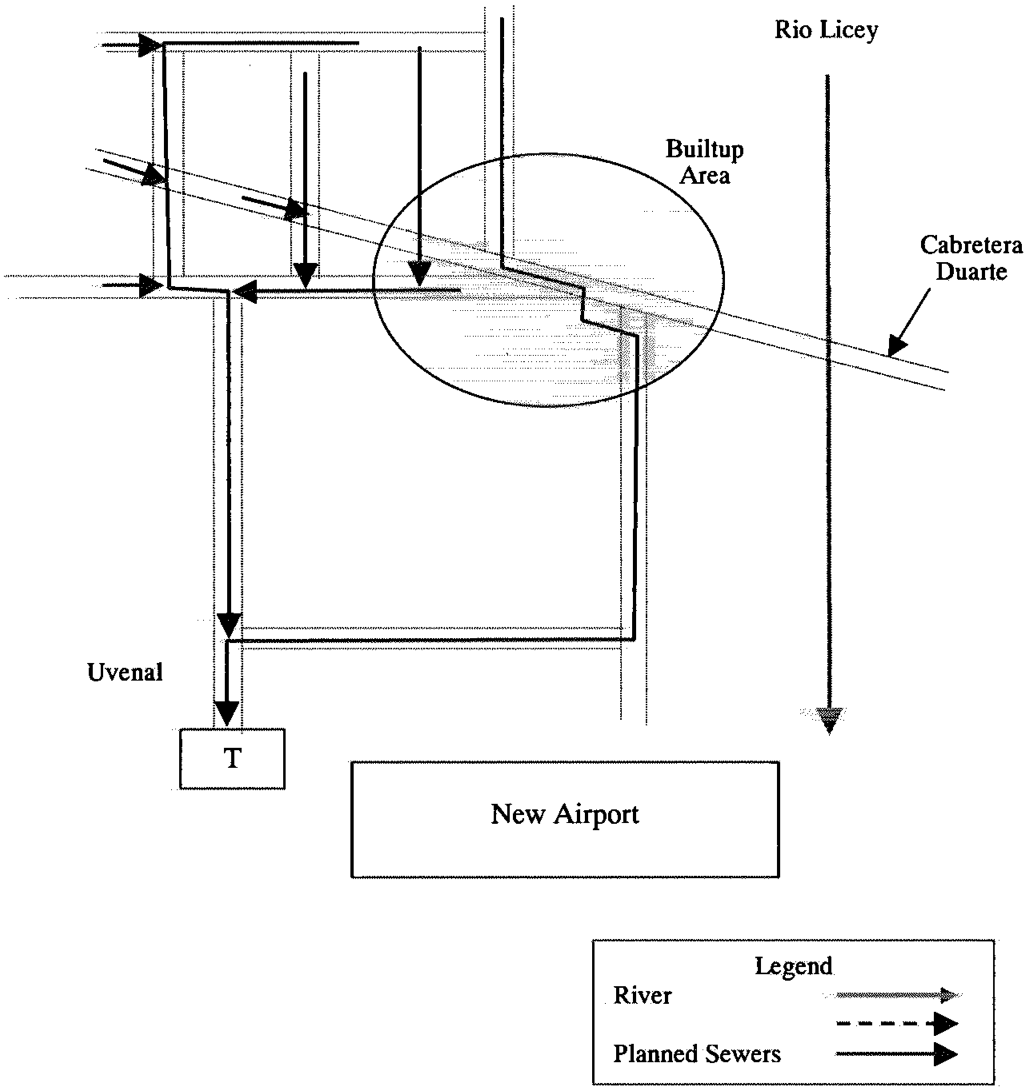
THE STUDY ON THE IMPROVEMENT OF SEWERAGE SYSTEM AND ENVIRONMENT IN THE CITY OF SANTIAGO

JAPAN INTERNATIONAL COOPERATION AGENCY

Figure 10.3
Outline of Cienfuegos and Los Salados Wastewater Collection System



THE STUDY ON THE IMPROVEMENT OF SEWERAGE SYSTEM AND ENVIRONMENT IN THE CITY OF SANTIAGO	Figure 10.4 Outline of Tamboril Wastewater Collection System
JAPAN INTERNATIONAL COOPERATION AGENCY	



THE STUDY ON THE IMPROVEMENT OF SEWERAGE SYSTEM AND ENVIRONMENT IN THE CITY OF SANTIAGO	Figure 10.5 Outline of Licey Wastewater Collection System
JAPAN INTERNATIONAL COOPERATION AGENCY	