as the groundwater infiltration, the amount of 4,320 m^3/d is equivalent about 15% of the maximum daily flow of the domestic, commercial, and institutional wastewater of 28,800 m^3/d.

In addition, when the additional wastewater is applied to the method in the Romanian Standards to estimate the groundwater infiltration, the equivalent length of sewers installed under groundwater table can be calculated as follows:

Total length of sewers installed: 90 km

Unit groundwater infiltration: 0.5 to 1.0 L/s/km of sewers installed (Romanian Standards)

The additional wastewater recorded: 50 L/s (4,320 m^3/d)

Length of sewers installed under the groundwater table:

50 L/s = (0.5 to 1.0) L/s/km x Equivalent Length

Equivalent Length of sewers: 100 km to 50 km

The calculated length of sewers are 55% to 110% of the total length of sewers, in other words, more than half of the sewers can be under the groundwater table.

According to SC ACET SA, the source of groundwater infiltration can be identified. Taking into account that about 20% of the recorded additional wastewater, 20 L/s, will be reduced by a rehabilitation program of sewer pipes and joints to protect groundwater infiltration, totally 30 L/s (2,592 m^3/d), 60% of the recorded wastewater of 50 L/s, is proposed for the design flow of groundwater infiltration.

3.5 SUMMARY OF DESIGN WASTEWATER FLOW

In summary, the design flows of domestic, commercial, institutional and industrial wastewater is combined and summarized in the table below.

Table AII.1.12 Summary of the Design Flow

(unit: m^3/d)

Wastewater	Average Daily	Maximum Daily	Maximum Hourly	Remarks
Domestic, commercial and Institutional Wastes	26,100	28,800	36,000	
Industrial Wastes	8,000	10,700	14,100	
Groundwater Infiltration	2,600	2,600	2,600	
Total	36,700 =>37,000	42,100 =>43,000	52,700 =>53,000	

4 WASTEWATER CHARACTERISTICS

4.1 PRESENT WASTEWATER CHARACTERISTICS

JICA Study Team conducted a wastewater quantity and quality survey during February to March in 1999. The samples were taken at two sites: one is a manhole at 1848 Street and another one is the Pumping Station (at 14 Noiembrie Street). The resulted four water quality items: BOD_5, SS, T-N, and T-P are presented in *Table AII.1.13* and *Figures AII.1.3 and AII.1.4*.

Those concentrations are varied as shown in the figures, a weighed average concentration for each parameter was calculated and presented in *Table AII.1.13*. Because residential area is the predominant in the service area of the sampling point No.1 (manhole at 1848 street), the samples were expected typical examples of domestic wastewater. But the very high concentration in SS observed in the surveys as shown in *Figure AII.1.3*. This indicates some industrial wastewater is also included in the wastewater.

Table AII.1.13 Results of Wastewater Quality Surveys (24 hours, one sample every 3 hours)

Parameters	Sampling Point No.1		Sampling Point No.2 (Pumping Station)		Remarks
	Range	Weighted Average	Range	Weighted Average	
BOD_5 conc. (mg/L)	28 - 136	95	11 - 91	38	
SS conc. (mg/L)	103 - 1,612	*187	37 - 195	83	
T-N conc. (mg/L)	20.8 - 39.9	31.4	7.5 - 17.8	12.6	
T-P conc. (mg/L)	0.88 - 3.07	2.24	0.5 - 1.62	1.09	

Note: * in this case, the highest measured value of 1,612 mg/L was excluded

The pollutant loads were estimated and summarized in *Table AII.1.14*.

Table AII.1.14 Estimated Pollutant Loads based on Wastewater Quantity and Quality Surveys

Sampling Location	Average Flow (m^3/d)	Weighted Average Concentration (mg/L)				Pollutant Loads (kg/d)				Remarks
		BOD_5	SS	T-N	T-P	BOD_5	SS	T-N	T-P	
Point No.1	3,288	95	187	31.4	2.24	312	615	103.2	7.37	
Point No.2	8,208	38	83	12.6	1.09	312	681	103.4	8.95	

Data: JICA Study Team

As the wastewater measured at sampling point no.1 was considered the domestic origin, the per capita unit pollutant loads is calculated and the result is shown in *Table AII.1.15*. The per capita wastewater generation is 598 L/capita/day (lpcd) . The unit loads are as high as 57 g/capita/d as of BOD_5 and 112g/capita/d as of SS. The results imply that some industrial wastewaters are included.

Table AII.1.15 Estimated Per Capita Unit Loads and Generation Rate of Domestic Wastewater

	Sampling Point No.1	Remarks
Average Flow (m3/d)	3,288	
Service Population *	5,500	
Per Capita Wastewater Generation (lpcd)	598	
Loads (kg/d)		
BOD_5	312	
SS	615	
Total Nitrogen (T-N)	103.2	
Total Phosphorus (T-P)	7.37	
Per Capita Unit Loads (g/capita/d)		
BOD_5	57	
SS	112	
Total Nitrogen (T-N)	18.8	
Total Phosphorus (T-P)	1.34	

Note: * shows that the service population is based on the information provided by SC ACET SA

4.1.2 INDUSTRIAL WASTEWATER

Table AII.1.16 shows quality data of BOD_5 and SS obtained through a questionnaire survey conducted by JICA Study Team with cooperation of SC ACET SA. Only the wastewater originated from food processing industries are analyzed to calculate the average quality. The resulted concentration is BOD_5 : 161 mg/L and SS: 91 mg/L, respectively.

4.2 DESIGN INFLUENT QUALITY

4.2.1 INTRODUCTION

Design wastewater quality is used as the basis for evaluation of effects of wastewater treatment as well as for making design of wastewater treatment facilities. As the design wastewater quality, influent quality and treated quality shall be determined. The latter quality, treated water quality is regulated by the Romanian Effluent Standards, as shown in *Table AII.1.17*. The detailed discussion on the treated wastewater quality for the design will be conducted in later opportunities when an appropriate wastewater treatment method is studied and proposed in a meeting.

Table AII.1.17 Major Effluent Quality Standards to Public Receiving Water Bodies

No.	Quality Parameters	Units	Max. Admissible	Methods of Analysis
	A. PHYSICAL PARAMETER			
1.	Temperature	°C	30°C	-
	B. CHEMICAL PARAMETER			
2.	Hydrogen ion concentration (pH) For Danube River	Unit pH	6.5 – 8.5 6.5 – 9.0	STAS 8619/3-90
3.	Total Suspended Solids	mg/dm^3	60.0	STAS 6953-81
4.	Biochemical Oxygen Demand (BOD_5)	mg/dm^3	20.0	STAS 6560-82
5.	Chemical Oxygen Demand (COD-Mn)	mg/dm^3	40.0	STAS 9887-74
6.	Chemical Oxygen Demand (COD-Cr)	mg/dm^3	70.0	STAS 6954-82
7.	Ammonium Nitrogen (NH_4^+-N)	mg/dm^3	2.0	STAS 8683-70
8.	Total Nitrogen (N)	mg/dm^3	10.0	STAS 7312-83
9.	Nitrates (NO_3^-)	mg/dm^3	25.0	STAS 8900/1-71
10.	Nitrites (NO_2^-)	mg/dm^3	1.0	STAS 8900/2-71
11.	Sulfides (as H_2S)	mg/dm^3	0.1	STAS 7510-66
12.	Sulphites (SO_3^{2-})	mg/dm^3	1.0	STAS 7661-89
13.	Phenols (C_6H_5OH)	mg/dm^3	0.05	STAS 7167-92
14.	Oil and Fats	mg/dm^3	5.0	STAS 7587-66
16.	Phosphates (PO_4^{3-})	mg/dm^3	4.0	STAS 10064-75
17.	Total phosphorus (P)	mg/dm^3	1.0	STAS 10064-75
C. BACTERIOLOGICAL PARAMETER				
42.	Total coliform (MPN)	Nr/100 cm^3	1 mil	STAS 3001-91
43.	Fecal coliform (MPN)	Nr/100 cm^3	10,000	STAS 3001-91
44.	Fecal streptococci (MPN)	Nr/100 cm^3	5,000	STAS 3001-91

Source: ORDER No. 730/1997, Norms for establishing the limits of pollutants in the wastewater before to be discharged into water resources, NTPA 001/1997

4.2.3 DESIGN INFLUENT QUALITY

The design influent quality, especially BOD_5, SS, T-N, and T-P is determined taking into consideration the present wastewater concentrations, pollutants loads, and data and information available from other references.

(1) Domestic, Commercial and Institutional Wastewater

Since any appropriate data was not obtained by the wastewater quality and quantity surveys for the Tulcea City, the following per capita unit loads is used to estimate the influent quality of domestic, commercial and public wastewater. It is assumed that the per capita loads include those for commercial and institutional pollutant loads at 30% of the domestic origin and the total per capita loads are increased about 30% within the target year. The influent quality is calculated as shown in *Table AII.1.18.*

Table AII.1.18 The Design Influent Quality of Domestic, Commercial, and Institutional Wastewater

Quality Parameter	Planned Service Population	Per Capita Loads (g/capita/d)	Loads (kg/d)	Design Average Flow (m^3/d)	Influent Quality (mg/L)	Remarks
BOD_5	73,000	44	3,212	26,100	123	
SS		51	3,723		140	
T-N		7.7	562		22	
T-P		1.01	74.0		2.8	

(2) Industrial Wastewater

The listed nine factories are categorized by their products and the present industrial wastewater discharges by product categories are summarized as shown in *Table AII.1.19*.

Table AII.1.19 Present Industrial Wastewater Discharges by Product Category

Category	Present Discharge Flow (m^3/d)	Share (%)	Remarks
Food Processing	583	4	Meat products, dairy products, fish, vegetables and fruits, etc.
Metal Products	13,422	89	Cast iron, bronze, aluminum products etc.
Machinery	906	6	Ship building
Others	150	1	
Total	15,061	100	

The design discharge flow to the sewerage system is set as shown in *Table AII.1.20*. The design discharge flow is calculated by the design flow of 5,700 m^3/d multiplied with the share of each category. JICA Study Team proposes the share shown in the table.

Table AII.1.20 Design Industrial Wastewater Discharge Flow by Categorized Factories

Category	Share (%)	Design Discharge Flow (m^3/d)	Remarks
Food Processing	53	3,000	
Metal Products	30	1,700	
Other Manufactures	17	1,000	
Total	100	5,700	Design Average Flow

For the design purpose, the industrial wastewater quality to be discharged by each category is proposed as shown in *Table AII.1.21*. The quality is set taking into account the present data available, the maximum permissible level set forth in the National Effluent Quality Standards for the Wastewater Discharge to Public Wastewater Systems as shown in *Table AII.1.22*, and some typical values for each category in references.

Table AII.1.21 Design Industrial Wastewater Characteristics Classified by Product Category

Category	Quality Parameters (mg/L)				Remarks
	BOD_5	SS	T-N	T-P	
Food Processing	300	200	40	10	
Metal Products	80	100	10	5	
Other Manufactures	100	100	20	2	
Others	100	200	5	1	

Table AII.1.22 Major Permissible Effluent Quality Standards for the Wastewater Discharged into Public Wastewater Systems

No.	Quality Parameter	Units	Permissible Values	Methods of Analysis
1.	Temperature	°C	40°C	-
2.	Hydrogen ion concentration (pH)	-	6.5 – 8.5	STAS 8619/3-90
3.	Suspended Solids	mg/dm^3	300	STAS 6953-81
4.	BOD_5	mg/dm^3	300	STAS 6560-82
5.	COD-Cr	mg/dm^3	500	STAS 6954-82
6.	Ammonium Nitrogen (NH_4^+-N)	mg/dm^3	30	STAS 8683-70
7.	Total Phosphorus (as P)	mg/dm^3	5.0	STAS 10064-75

Source: Norms regarding the discharge conditions of wastewater into sewerage, NTPA 002/1997

The maximum permissible concentrations of BOD_5 and SS are set at 300 mg/L as the same as the national effluent quality standards for the wastewater discharge to public wastewater systems as shown in *Table AII.1.22*. However, regarding the concentration of total nitrogen and total phosphorus, the national effluent standards are not applied. Because the national standards do not provide any maximum permissible concentration of total nitrogen but that of ammonium nitrogen of 30 mg/L and provide that of total phosphorus of 5.0 mg/L.

The design loads from the point source are estimated as shown in *Table AII.1.23*; the design discharge flows multiplied with the concentrations. The average concentration is estimated, the total loads are divided with the total flow: BOD_5 of 199 mg/L, SS of 153 mg/L, T-N of 28 mg/L, and T-P of 7.1 mg/L.

Table AII.1.23 Design Quality of Industrial Wastewater of Point Source

Category	Design Flow (m^3/d)	Concentration (mg/L)		Loads (kg/d)		Remarks
		BOD_5	SS	BOD_5	SS	
Food Processing	3,000	300	200	900	600	
Metal Products	1,700	80	100	136	170	
Other Manufactures	1,000	100	100	100	100	
Total	5,700			1,136	870	
Average Concentration (mg/L)		199	153			

Table AII.1.23 (continued) Design Quality of Industrial Wastewater of Point Source

Category	Design Flow (m^3/d)	Concentration (mg/L)		Loads (kg/d)		Remarks
		T-N	T-P	T-N	T-P	
Food Processing	3,000	40	10	120	30.0	
Metal Products	1,700	10	5	17	8.5	
Other Manufactures	1,000	20	2	20	2.0	
Total	5,700			157	40.5	
Average Concentration (mg/L)		28	7.1			

The design quality of overall industrial wastewater is estimated as shown in *Table AII.1.24*. In the table, the design quality of industrial wastewater originated from non-point source is assumed to be the same as the domestic, commercial and institutional wastewater, i.e. BOD_5 of 123 mg/L, SS of 140 mg/L, T-N of 22 mg/L, and T-P of 2.8 mg/L. The design quality of overall industrial wastewater is estimated as follows: BOD_5 of 177 mg/L, SS of 149 mg/L, T-N of 26 mg/L, and T-P of 6.0 mg/L.

Table AII.1.24 Design Quality of Industrial Wastewater

Source	Design Flow (m^3/d)	Concentration (mg/L)		Loads (kg/d)		Remarks
		BOD_5	SS	BOD_5	SS	
Point Source	5,700	199	153	1,136	870	
No-point Source	2,300	123	140	283	322	
Total	8,000			1,419	1,192	
Average Concentration (mg/L)		177	149			
		T-N	T-P	T-N	T-P	
Point Source	5,700	28	7.1	157	40.5	
No-point Source	2,300	22	2.8	51	6.4	
Total	8,000			208	46.9	
Average Concentration (mg/L)		26	6.0			

Combine the design quality of domestic, commercial, and institutional wastewater shown in *Table AII.1.18* with that of industrial wastewater discharged to the public sewerage system shown in *Table AII.1.24*, the overall influent quality to the wastewater treatment plant is estimated as shown in *Table AII.1.25*.

Consequently, the design influent quality is BOD_5: 130 mg/L, SS: 140 mg/L, T-N: 20 mg/L, and T-P: 3.5 mg/L.

Table AII.1.25 Design Influent Qualities

Wastewater	Design Flow (m^3/d)	Loads (kg/d)		Concentration (mg/L)		Remarks
		BOD_5	SS	BOD_5	SS	
Domestic, Commercial, and Institutional	26,100	3,212	3,723	123	140	
Industrial	8,000	1,419	1,192	177	149	
Groundwater	2,600	0	0	0	0	
Total	36,700	4,631	4,915			
Average Concentration (mg/L)				126 => 130	134 => 140	

Table AII.1.25 (continued) Design Influent Quality

Wastewater	Design Flow (m^3/d)	Loads (kg/d)		Concentration (mg/L)		Remarks
		T-N	T-P	T-N	T-P	
Domestic, Commercial, and Institutional	26,100	562	74.0	22	2.8	
Industrial	8,000	208	46.9	26	6.0	
Groundwater	2,600	0	0	0	0	
Total	36,700	770	120.9			
Average Concentration (mg/L)				21 => 20	3.3 => 3.5	

Table AII.1.8 Major Manufactures and Companies Discharging Industrial Wastewater in Tulcea City

No.	database no.	Comapany Name	Category Code	Water Consumption (1998) Yearly (m^3/year)	Monthly (m^3/month)	Daily (/30d) (m^3/d)	Wastewater Discharge to the sewers (m^3/d)	Discharge to Danube River (1998) Yearly (m3/yr)	Monthly (m3/month)	Daily(/30d) (m3/d)	Total Discharge (m^3/d)	Discharge Ratio (%)
9	3	SC TABCO SA	1211	117,920	9,827	328	328				328	
8	4	SC DELTALACT SA	1212	42,826	3,569	119	119				119	
7	11	SC TULCO SA	1221	28,169	2,347	78		16,000	1,333	44	44	
3	10	ECODELTA SA	1229	123,848	10,321	344		-			0	
2	9	SC DALLCO Dunarea SA	1231	141,181	11,765	392		33,000	2,750	92	92	
		Food Processing		336,024	37,829	1,261	447	49,000	4,083	136	583	4%
1	8	BBG ALUM SA	2733	595,706	49,642	1,655		4,822,000	401,833	13,394	13,394	
4	7	FEROM SA	2741	426,906	35,576	1,186		10,000	833	28	28	
		Metal Products		1,022,612	85,218	2,841	0	4,832,000	402,666	13,422	13,422	89%
6	5	S.N.Tulcea S.A.	3141	375,698	31,308	1,044		326,000	27,167	906		
		Machinery		375,698	31,308	1,044	0	326,000	27,167	906	906	6%
5	6	FRIGORIFER SA	n.a.	20,682	1,724	57		54,000	4,500	150		
		Others		20,682	1,724	57	0	54,000	4,500	150	150	1%
		TOTAL		1,755,016	156,079	5,203	447	5,261,000	438,416	14,614	15,061	

Table AII.1.16 Industrial Wastewater Characteristics at Major Manufactures in Tulcea City

No.	database no.	Comapany Name	Category Code	Wastewater Discharg to the Sewers (m^3/d)	Discharge to Danube River (1998) Yearly (m^3/yr)	Monthly (m^3/month)	Daily(/30d) (m^3/d)	Total Discharge (m^3/d)	Discharge Ratio (%)	Effluent Quality BOD$_5$ Conc. (mg/L)	BOD$_5$ Loads (kg/d)	BOD$_5$ Ave. Conc. (mg/L)	SS Conc. (mg/L)	SS Loads (kg/d)	SS Ave. Conc. (mg/L)
9	3	SC TABCO SA	1211	328				328		59	19		46	15	
8	4	SC DELTALACT SA	1212	119				119		436	52		203	24	
7	11	SC TULCO SA	1221		16,000	1,333	44	44		103	5		44	2	
3	10	ECODELTA SA	1229		-			0		0	0		0	0	
2	9	SC DALLCO Dunarea SA	1231		33,000	2,750	92	92		201	18		134	12	
		Food Processing		447	49,000	4,083	136	583	4%		94	161		53	91
1	8	BBG ALUM SA	2733		4,822,000	401,833	13,394	13,394		N.A.			N.A.		
4	7	FEROM SA	2741		10,000	833	28	28		N.A.			N.A.		
		Metal Products		0	4,832,000	402,666	13,422	13,422	89%						
6	5	S.N.Tulcea S.A.	3141		326,000	27,167	906			0			243		
		Machinery		0	326,000	27,167	906	906	6%						
5	6	FRIGORIFER SA	n.a.		54,000	4,500	150			N.A.			N.A.		
		Others		0	54,000	4,500	150	150	1%						
		TOTAL		447	5,261,000	438,416	14,614	15,061							

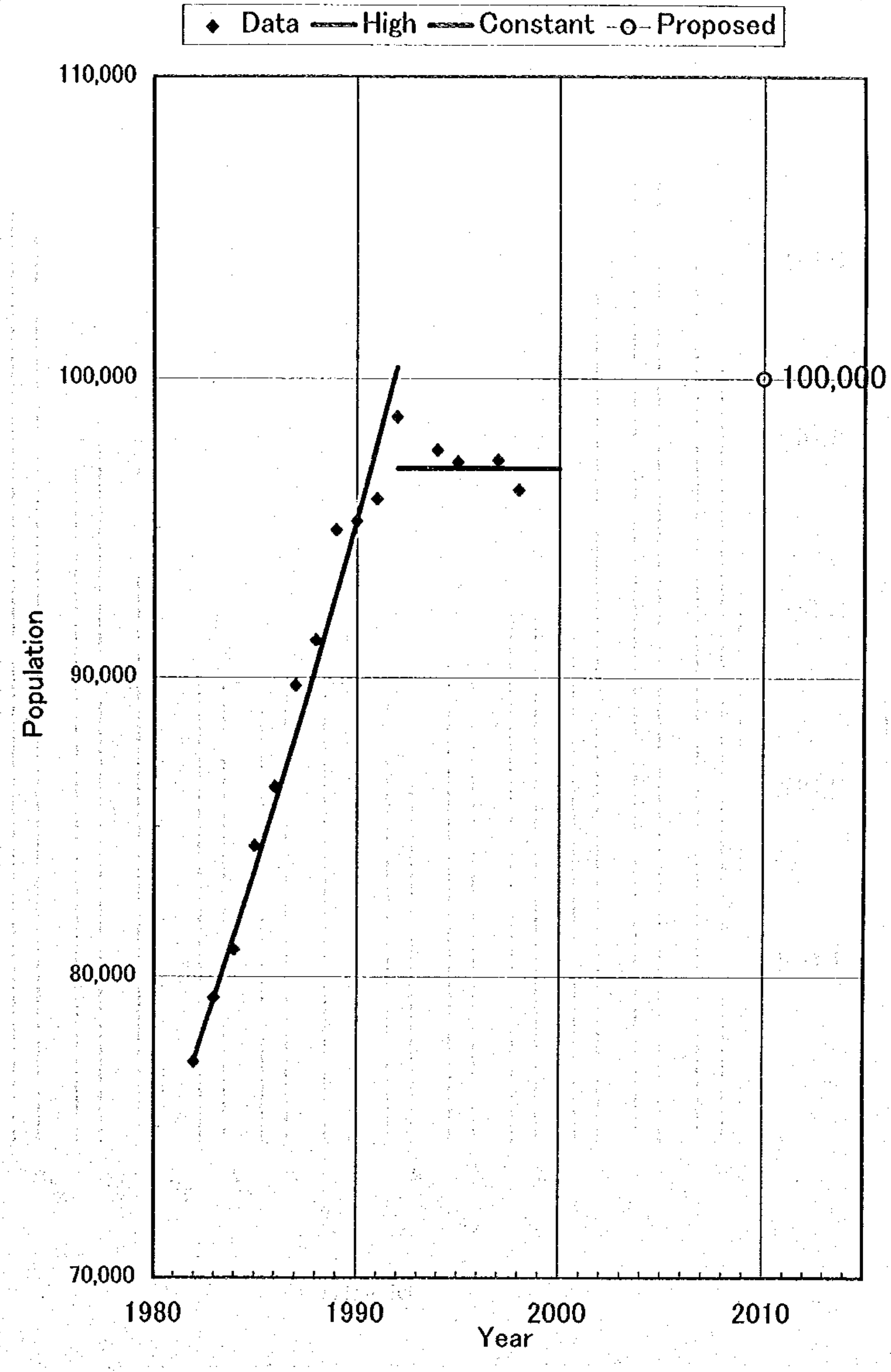


Figure AII.1.1	Population Projection for Tulcea City

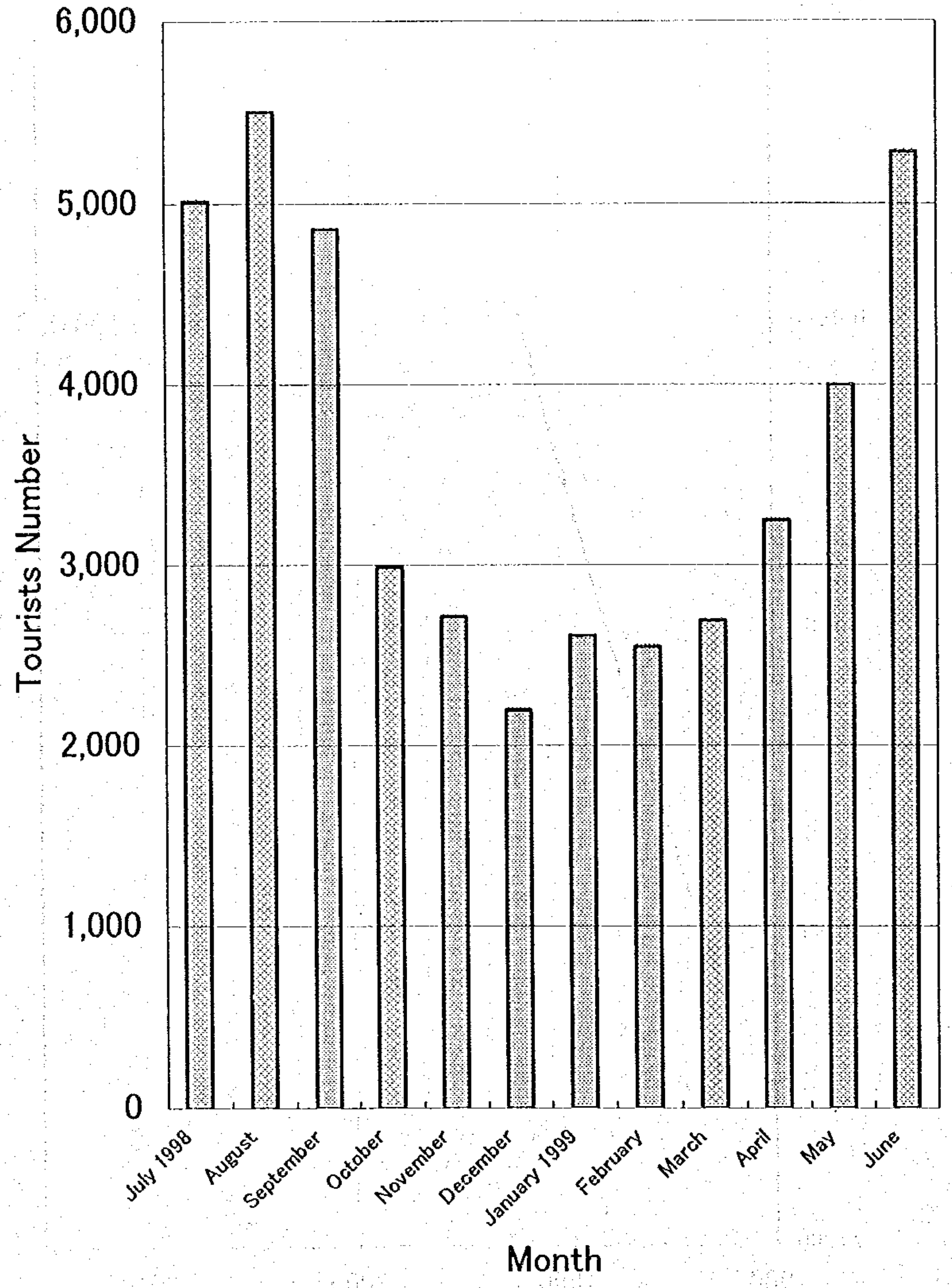
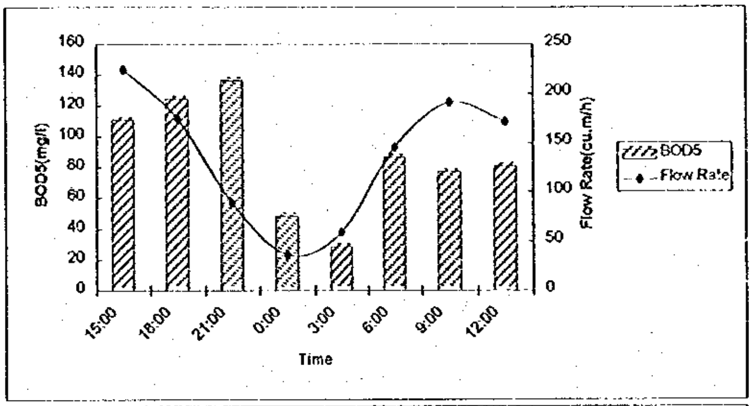
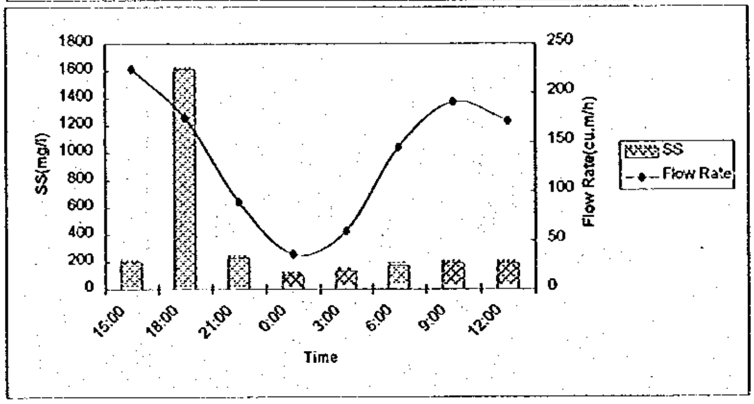
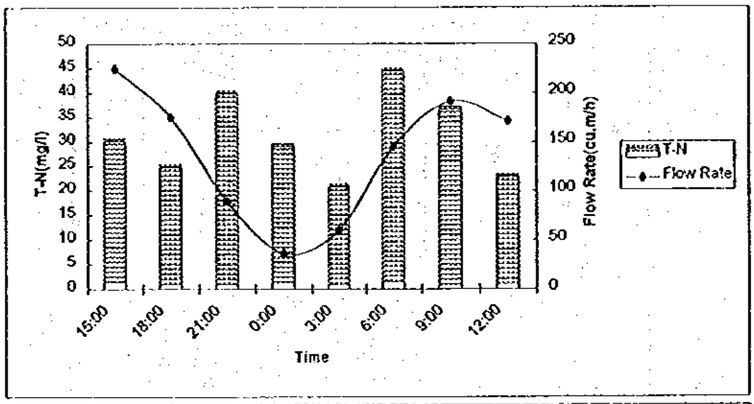


Figure AII.1.2	Tourist Number Stayed in Three Major Hotels in Tulcea City

City: Tulcea
Sampling Point: No.1







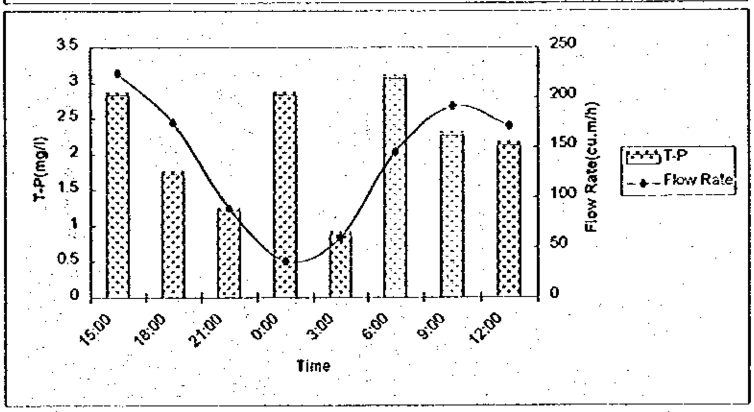


Figure AII.1.3	Result of Wastewater Quality Survey at 1848 Street in Tulcea City

City: Tulcea
Sampling Point: No.2

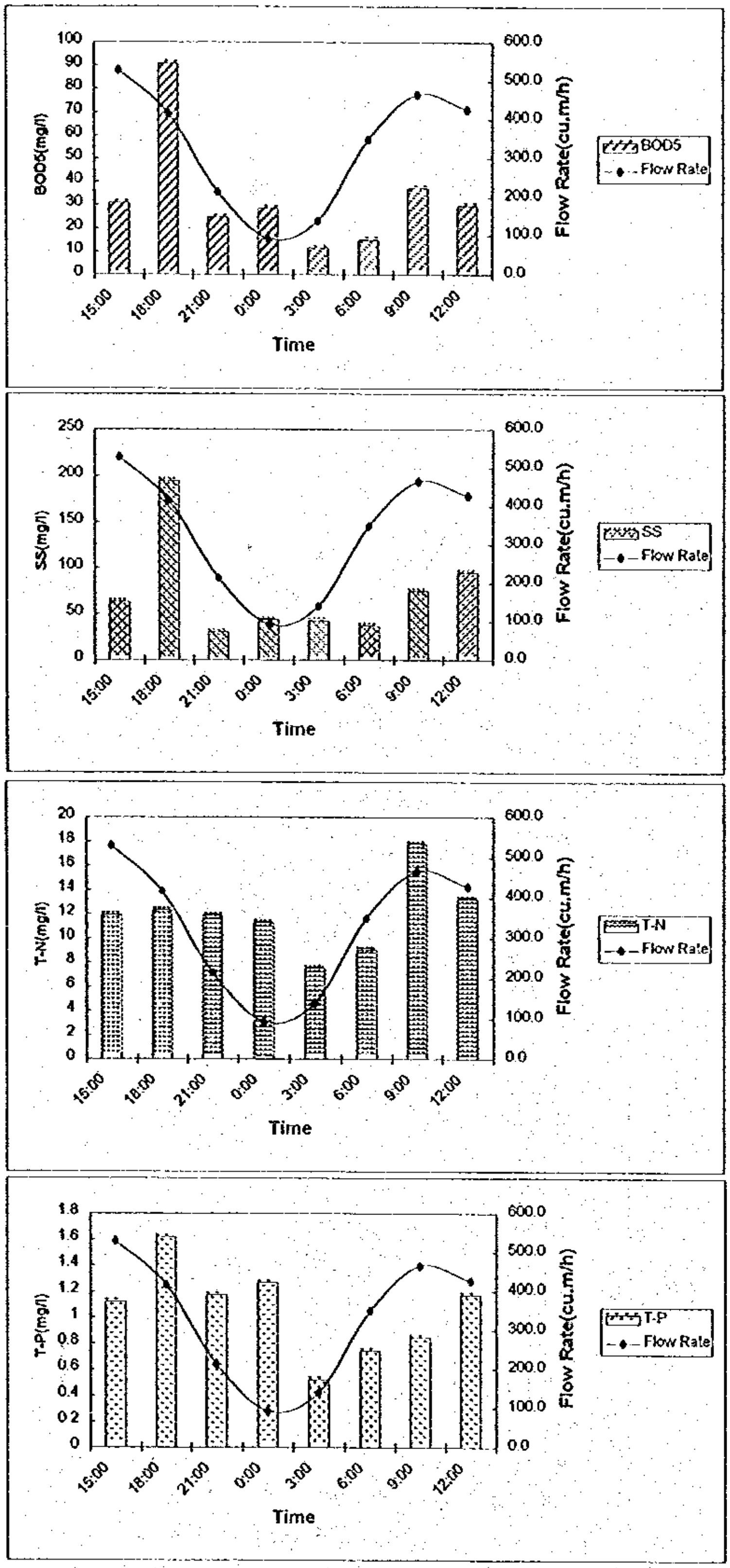


Figure AII.1.4	Result of Wastewater Quality Survey at the Pumping Station in Tulcea City

APPENDIX-2 INTERCEPTOR SYSTEM

1. EXISTING OUTFALLS

The Tulcea sewerage system is in principle of a separate system to collect the wastewater and stormwater separately through sanitary and storm sewer systems, respectively, although in some areas part of the stormwater inflows to the sanitary sewers. All the collected wastewater through the sewer networks is currently being discharged into the Danube River through seven (7) outfalls. Sewerage system in Tulcea is shown in *Figure AII.2.1 and AII.2.2*

Among the sewer outfalls, No.5 and No.6 outfalls, which locate near the poposed WWTP site, discharge major portion of the wastewater from the city area conveyed by two (2) major sewers. The proposed wastewater treatment plant (WWTP) site is located near the outfalls. The No.5 and No.6 outfall pipelines were laid in 1970 and 1981, respectively. The two pipelines of 1,000 mm in diameter, having the same invert elevation, are laid almost in parallel starting from the city center to the Danube River, but are not connected to each other. The outfalls are located under the river water level and are pressured.

The outfalls of No. 1, No. 2, No. 3 and No.4 discharge wastewater to the Danube River in the central part of the Tulcea City. The outfall No. 7 discharges wastewaer generated only at the water purification plant, so that the plant wastewater is to be managed by the plant itself. As such, the plant wastewater is excluded from the present study.

Flows of each outfall was estimated considering the sewer service area and landuse as shown in *Table AII.2.1*, *Table AII.2.2* and *Figure AII.2.1* and *AII.2.3*.

2. PROPOSED INTERCEPTOR SYSTEM

2.1 COLLECTION SYSTEM PLANNING

It is planned that the two major pipelines receive all the wastewater generated in the City and lead it to the proposed WWTP located near the existing outfalls No. 5 and No.6. The flow capacity of the one interceptor sewer at the velocity of 1 m/s is, 1 m x 1 m x 3.14 / 4 x 1.0 m/s = 0.785m^3/sec, which is more than the maximum hourly flow of 0.614 m^3/sec, and that the capacity of the two pipelines is sufficient to flow the maximum hourly wastewater flow of 53,000 m^3/day or 0.614 m^3/sec to the WWTP without causing any hydraulic hindrance.

It is necessary to provide the wastewater collection system to lead all the sanitary wastewater into the two major interceptors. At present, there is a sewer along the Street Isaccei, which is crossing four outfall sewers, i.e. No. 1, 2, 3 and 4. The sewer is to collect wastewater from two areas and lead it to the existing pumping station (SP3) that pumps the wastewater up to a nearby manhole.

The manhole top elevation is about +16 m M.W.L. whereas the average water level is about +15 m M.W.L. One sewer has been installed from the manhole to an old existing major sewer, which leads the wastewater to the outfall No. 5.

The wastewater collecting system from the four outfalls to the major interceptors is planned to use the existing sewer along the street Isaccei by diverting the wastewater flow from the existing four outfalls to the sewer.

2.2 CONNECTION OF EXISTING OUTFALLS TO NEW INTERCEPTORS

The existing sewer system layout in the subject area is shown in *Figure AII.2.4*. To establish an appropriate interceptor sewer system, two alternative diverting systems are studied. Alternative 1 is to use the presently working pumping station SP3 whereas Alternative 2 is planned to use the stormwater pumping station SP3 which is named as S-SP3 in this study, as shown in *Figure AII.2.5.* At present, the stormwater pumping station S-SP3 is not being operated, as the flooding condition has not been so serious. The S-SP3 does not have any equipment but only the structure.

Sewer calculation sheet and profile of an existing sewer along the Street Isaccei are shown in *Table AII.2.3* and *Fig AII.2.6.* It has been observed that the wastewater does not flow smoothly from the sewer TI2 to sewer TI5, because the invert elevations of TI5 and TI6 are higher than the invert elevation of the upstream sewer as it can be seen from the profile. For these reasons, the existing sewer from TI5 needs to be replaced.

The ground elevations at the pumping station SP3 for the wastewater and at the pumping station S-SP3 for stormwater are +6.25 m and +4.5 m M.W.L. respectively. Low water levels at SP3 and S-SP3 are fixed to +1.75 m and 0.0 m M.W.L. respectively, since both stuructures are of the same size and the low water level is 4.5 m lower than the ground level, which is equal to ground floor level.

The flow calculation sheets for Alternative 1 and 2 are shown in *Table AII.2.4 and AII.2.5* respectively. Longitudinal sewer profiles for Alternative 1 and Alternative 2 are shown in *Figure AII.2.7* and *AII.2.8,* respectively. From these tables and figures, it was revealed that Alternative 1 cannot be connected the sewer to the existing pumping station SP3, as the sewer line TI6 invert elevation of 0.966 m M.W.L. is lower than SP3's low water level of +1.75 m M.W.L. (HWL: 4.25 m), thus Alternative 1 is considered unrealistic. In case of Alternative 2, the TI5's invert elevation of +1.135 m M.W.L. is higher than the low water level of 0.00 m (HWL: 2.5 m) in S-SP3 for stormwater. In view of these conditions, Alternative 2 is selected as the appropriate interceptor system.

Based on the flow calculation sheet of *Table AII.2.5,* the existing sewer from TI4 needs to be replaced to have steeper slope of 2.6 o/oo instead of the present slope 0.7 o/oo, as the capacity of existing TI4 is not enough to carry maximum hourly flow. Layout of planned interceptor for Alternative 2 is shown in *Figure AII.2.9.*

2.3 PUMPING STATION

Pump equipment are to be installed in the existing stormwater pumping station S-SP3 to send wastewater to a manhole tank. The dimension of structure is 9.0 m diameter, with effective depth of 2.5 m (HWL: 2.5 m, LWL: 0.0 m) and effective volume of about 159 m^3. The effective volume can retain about 9 minutes of the maximum hourly flow of 0.281 m^3/s.

The estimated wastewater flow is 0.281 m^3/s and the actual head is about 15 m, because low water level of the S-SP3 and high water level of the manhole are 0.0 m and +15 m, respectively. The pressure pipe diameter is calculated to be 400 mm at the velocity of about 2.2 m/s. The total head losses is estimated to be about 5 m since the total friction losses is about 4.2 m, so that the required overall head is estimated to be 30 m.

The required number of pumps is three with one standby pump. The planned pump equipment is as follows:

- Capacity	:	0.15 m^3/s for each pump
- TDH	:	30 m
- Number of pumps	:	3 (1 standby)

2.4 CONFIRMATION OF WATER HEADS IN SEWER PIPES

As the two outfall pipelines are under the hydraulic pressure, hydraulic heads of the existing major sewer pipelines and the proposed sewer pipes have been checked.

The existing water heads of major pipelines to send wastewater are verified by checking the friction losses and location of the lowest house connection. Elevation of the lowest house connection pipe is placed higher than the required head to discharge the wastewater through the two main sewers under pressure even in the Danube River water is at the high water level of +5.0 m M.W.L. These are shown in *Table AII.2.6.*

The future water heads of major pipelines to send wastewater after the project implementation are also confirmed in the same manner and it is verified as shown in *Talbe AII.2.7.*

2.5 PROPOSED FACILITIES

Planned facilities comprise connection sewers, interceptor sewers, manholes and valves. These are described in *Table AII.2.8.* Major features of the facilities are as follows:

(1) Interceptor sewer

Interceptor sewer system comprises two major sewers, one is the major interceptor near the proposed WWTP and the other is in the central part of the City. The major interceptors consist of two sewers of 1,000 mm in diameter and 600 m long, which collect and send the wastewater to the proposed WWTP.

The other interceptor sewers in the City center is to replace and to newly install sewer along the Street Isaccei to S-SP3 with diameter: 600 mm and length: 87 m. Other sewers to be installed from the S-SP3 to a manhole are pipelines of 400 mm diameter and 285 m long, and sewers from the manhole tank to the major interceptors with 400 mm diameter and 173 m long. The sewer from the S-SP3 to the manhole tank is steel pipe as it is pressured.

Generally, the sewer construction is conducted by open cut method. The earth coverage of all installed sewers ranges from 1m to 3 m.

Typical sewer construction is shown in *Figure AII.2.10.*

(2) Connection sewers

As the Tulcea sewerage system is generally of separate system, it is considered that sanitary sewers do not contain stormwater. In order to intercept the sanitary wastewater being discharged from the outfalls No. 1, 2, 3 and 4, only connection sewers are to be installed without combined sewer overflow (CSO) regulators.

The connection sewers are to carry the maximum hourly flow from the existing manholes of the sewers to the interceptor sewer. Length of connection sewer is in general approximately 10 m and the earth coverage is from 1m to 3 m. Totally 4 connection sewers will be installed.

Typical sewer construction is shown in *Figure AII.2.10.*

(3) Manholes

Manholes are installed along the interceptor sewer at intervals of about 100 m, as the sewer diameter is less than 800 mm. They are also installed at the sewer and road junctions.

Totally 2 manholes will be installed along the interceptor sewers. One special manhole is installed at the terminal point of the sewer from the S-SP3.

Typical manhole structure is shown in *Figure AII.2.11*.

(4) Valves

Valves are to be installed for the major interceptors for two areas. One area is near the connection point from the manhole. Two valves of diameter 1,000 mm are to be installed there to control the flow for maintenance purpose. The other area is near the proposed WWTP site. One valve along No.5 old major interceptor has been installed near the Danube River. Hence, three valves of 1,000 mm diameter are to be installed to control the flows.

(5) Pump Equipment

Three pumps of 0.15 m^3/sec x 30 m TDH with accessories are to be installed at S-SP3.

(6) CSO Regulator

Combined sewer overflow (CSO) regulators is installed at main sewer. The CSO regulators let exceeding wastewater overflow from weirs to the existing outfalls. Typical structures of CSO regulators are shown in *Figure AII.2.12*.

3. WWTP OUTFALL SEWERS

A pressure outfall sewer is to be laid from the WWTP to the Danube River, crossing the riverbank. The sewer is under pressure from the WWTP chlorination chamber throughout the discharge point. When the Danube river water level becomes higher than a set water level, discharge-pumping station starts its operation to discharge the treated wastewater effluent to the river. The pumping station receives the treated wastewater from the chlorination chamber and discharges the water to the outfall sewers.

The outfall sewer diameter is determined to be 800 mm based on the calculated head losses and flow velocity in the sewer pipe. When the maximum hourly flow (53,000 m3/day = 0.614 m3/s) inflows to the outfall sewer, the velocity of flow in the sewer is about 1.2 m/s.

Earth coverage is planned based on the water level of chlorination chamber: 1.0 m, the water depth in the chlorination chamber: 4.0 m and the ground level at the chlorination chamber: + 2.0 m M.W.L. The sewer elevations at the invert and top are about -2.5 m and -1.5 m M.W.L., respectively, so the earth coverage is selected to be 3.5 m.

The outfall sewer starts near the chlorination chamber and the ends up at the discharge point in the River. The river bottom elevation at the end point is almost same as the sewer invert elevation of -2.5 m. Hence, the outfall sewer length is about 150 m.

Table AII.2.1 Flow of Each Outfall in Tulcea

Serial No.	Each Outfall	Landuse Area (ha)			No. of Population (person)			Flow (m3/s)
		Blocks	Individual House	Toal	Blocks	Individual House	Toal	
1		0.20	2.38	2.58	60	119	179	0.002
2		3.20	0.00	3.20	960	0	960	0.009
3		2.34	0.00	2.34	702	0	702	0.007
4		59.85	73.38	133.23	17,955	3,669	21,624	0.208
5-1	by Gravity East	18.27	0.00	18.27	5,481	0	5,481	0.053
5-2	Through SP0	6.44	0.00	6.44	1,932	0	1,932	0.019
5-3	Through SP1,2 and gravity	57.40	12.81	70.21	17,220	641	17,861	0.172
5-4	Through SP3 East	13.43	1.39	14.82	4,029	70	4,099	0.040
5-5	Through SP3 West	2.80	15.23	18.03	840	762	1,602	0.015
5-6	Through SP4	1.15	0.00	1.15	345	0	345	0.003
Total of 5		99.49	29.43	128.92				0.302
6		27.96	10.14	38.10	8,388	507	8,895	0.086
Total		193.04	115.33	437.29	57,912	5,768	63,680	0.614
Flow of Existing SP3								0.055
Flow of Planned SP3								0.281

Remark 1 Total hourly maximum flow to WWTP was estimated as follows and the detail is shown in wastewater planning,

Q(hourlymax) = 53000 m3/day
Q(hourlymax) = 0.614 m3/s

The total hourly Max. flow was uniformly divided for each outfall with the ratio of the service population.

The population was calculated by multiplying an area and the population density for the landuse of blocks and individual house respectively.

Population density of the blocks and individual house is as follows,

	Population Density (person/ha)	
	*1)	Applied Pop. Density
Blocks	298.9	300
Individual House	47.4	50

*1) Source: Tulcea City Population data in 1992

Remark 2 The mentioned population was calculated only to estimate the each outfall flow for the sewer designing , but not for the wastewater planning.

Table AII.2.2 Population Density Categorized in Landuse InTulcea

Land Use	Name of sub-distric	No. of District	Area (ha)	Population	Population Density
Individual Hous	Dealul Monumentul	2	31.5	2,917	93
	Platoul Morilor	11	122.5	7,836	64
	Dealul Mahmdiei	15	81.0	3,414	42
	Pacii	18	34.5	3,995	116
	Grivitei	14	28.5	2,093	73
	Eroilor	8	65.5	4,693	72
	Victoriei	7	23.5	2,630	112
	Plopiolor	21	161.0	243	2
	Vararie	33	-	774	-
	Mal Zaghen	20	44.0	937	21
	Parcelarea Noua	27	31.0	-	-
	Sub-total		623.0	29,532	47
Apartment	Bariera Isaccei	5	17.5	7,678	439
	Spitalului	6	42.0	10,997	262
	Babadag Centru	9	6.0	1,572	262
	Cazarmii	12	38.5	8,492	221
	Scolilor	13	17.0	3,332	196
	Babadag Intrare	16	23.0	6,759	294
	Cartier Sud	17	18.0	9,591	533
	Sub-total		162.0	48,421	299
Industry Area	Zona Ind. Est	22	129.0	4,881	38
	Santier Naval	23	116.0	3,834	33
	Islaz	24	85.0	200	2
	Zona Ind. Vest	25	118.0	3,872	33
	Depozite	26	62.0	1,681	27
	Sub-total		510.0	14,468	28
Animal Farm	Somova	32	103.0	196	2
	Cisla	31	-	-	-
	Viilor	29	-	-	-
	Sub-total		103.0	196	2
Green Area	Ciuperca	3	60.0	90	2
	Bididia	19	-	-	-
	Lac Zaghen	34	-	-	-
	Sub-total		60.0	90	2
Other Area		1	32.0	6,996	219
		10	22.5	6,731	299
		4	37.0	4,494	121
		28	15.0	-	-
	Sub-total		106.5	18,221	171
Total			1,564.5	110,928	71

No. of District	Name of sub-district	Area (ha)	Population	Population Density
1		32.0	6,996	219
2	Dealul Monumentului	31.5	2,917	93
3	Ciuperca	60.0	90	2
4		37.0	4,494	121
5	Bariera Isaccei	17.5	7,678	439
6	Spitalului	42.0	10,997	262
7	Victoriei	23.5	2,630	112
8	Eroilor	65.5	4,693	72
9	Babadag Centru	6.0	1,572	262
10		22.5	6,731	299
11	Platoul Morilor	122.5	7,836	64
12	Cazarmii	38.5	8,492	221
13	Scolilor	17.0	3,332	196
14	Grivitei	28.5	2,093	73
15	Dealul Mahmdiei	81.0	3,414	42
16	Babadag Intrare	23.0	6,759	294
17	Cartier Sud	18.0	9,591	533
18	Pacii	34.5	3,995	116
19	Bididia	-	-	-
20	Mal Zaghen	44.0	937	21
21	Ploplolor	161.0	243	2
22	Zona Ind. Est	129.0	4,881	38
23	Santier Naval	116.0	3,834	33
24	Islaz	85.0	200	2
25	Zona Ind. Vest	118.0	3,872	33
26	Depozite	62.0	1,681	27
27	Parcelarea Noua	31.0	-	-
28		15.0	-	-
29	Viilor	-	-	-
31	Cisla	-	-	-
32	Somova	103.0	196	2
33	Vararie	-	774	-
34	Lac Zaghen	-	-	-
	Total	1,564.5	110,928	71

Remark
Source: Tulcea City
Population is in 1992.

Table AII.2.3 Computations of Existing Sewer along Street Isaccei to SP3 in Tulcea

Line No.	Line No. of	Sewer Length (m)		Max. Flow (m3/s)			Sewer Line				Sewer Invert Elevation (m)		Ground Elevation (m)		Earth Covering (m)	
	Lower Sewer	Increment	Total	Sewage	Infilt.	Total	Diameter (mm)	Slope (o/oo)	V (m/s)	Cap. (m3/s)	Upper end	Lower end	Upper end	Lower end	Upper end	Lower end
TI0	TI1	390	390	0.040	0.000	0.040	300	1.7	0.667	0.047	3.840	3.180	5.240	4.880	1.100	1.400
No.1	TI1	-	-	0.002	0.000	0.002	350	-	-	-	3.400	3.400	-	4.900	-	1.150
TI1	TI2	482	872			0.040	400	2.3	0.939	0.118	3.180	2.070	4.900	4.270	1.320	1.800
No.2	TI2	-	-	0.009	0.000	0.009	900x600	-	-	-	2.650	2.650	4.450	4.270	1.200	1.020
No.3	TI2	-	-	0.007	0.000	0.007	500	-	-	-	2.650	2.650	4.450	4.270	1.300	1.120
TI2	TI3	210	1,082			0.040	400	3.3	1.125	0.141	2.070	1.370	4.270	4.250	1.800	2.480
No.5-5	TI3	-	-	0.015	0.000	0.015	400	-	-	-	-	2.550	-	4.250	-	1.300
TI3	TI4	17	1,099			0.055	600	2.4	1.257	0.355	1.370	1.330	4.250	4.030	2.280	2.100
No.4	TI4	-	-	0.208	0.000	0.208	500	-	-	-	2.860	2.860	4.360	4.030	1.000	0.670
TI4	TI5	75	1,174			0.055	600	0.7	0.679	0.192	1.330	1.280	4.030	4.550	2.100	2.670
TI5	TI6	22	1,196			0.055	600	-32.7	-	-	1.280	2.000	4.550	5.400	2.670	2.800
TI6	SP3	43	1,239			0.055	600	5.8	1.955	0.553	2.000	1.750	5.400	6.250	2.800	3.900

Table AII.2.4 Computations of Planned Interceptor along Street Isaccei to SP3 in Tulcea (Alt.1)

Line No.	Line No. of	Sewer Length (m)		Max. Flow (m3/s)			Sewer Line				Sewer Invert Elevation (m)		Ground Elevation (m)		Earth Covering (m)		Remark
	Lower Sewer	Increment	Total	Sewage	Infilt.	Total	Diameter (mm)	Slope (o/oo)	V (m/s)	Cap. (m3/s)	Upper en	Lower end	Upper en	Lower end	Upper en	Lower end	
TI0	TI1	390	390	0.040	0.000	0.040	300	1.7	0.667	0.047	3.840	3.180	5.240	4.880	1.100	1.400	*1)
No.1	TI1	-	-	0.002	0.000	0.002	300	-	-	-	3.400	3.400	-	4.900	-	1.200	*1)
TI1	TI2	482	872			0.042	400	2.3	0.939	0.118	3.180	2.070	4.900	4.270	1.320	1.800	*1)
No.2+3	TI2	-	-	0.016	0.000	0.016	300	-	-	-	2.650	2.650	4.450	4.270	1.500	1.320	*1)
TI2	TI3	210	1,082			0.058	400	3.3	1.125	0.141	2.070	1.370	4.270	4.250	1.800	2.480	*1)
No.5-6	TI3	-	-	0.015	0.000	0.015	400	-	-	-	-	2.550	-	4.250	-	1.300	*1)
TI3	TI4	17	1,099			0.073	600	2.4	1.257	0.355	1.370	1.330	4.250	4.030	2.280	2.100	*1)
No.4	TI4	-	-	0.208	0.000	0.208	400	-	-	-	2.860	2.860	4.360	4.030	1.100	0.770	*1)
TI4	TI5	75	1,174			0.281	600	2.6	1.107	0.313	1.330	1.135	4.030	4.550	2.100	2.815	*2)
TI5	TI6	22	1,196			0.281	600	2.6	1.107	0.313	1.135	1.078	4.550	5.400	2.815	3.722	*2)
TI6	SP3	43	1,239			0.281	600	2.6	1.107	0.313	1.078	0.966	5.400	6.250	3.722	4.684	*2)

Remark *1) : Using existing sewer
*2) : Replacement of existing sewer with new sewer

Table AII.2.5 Computations of Planned Interceptor along Street Isaccei to S-P3 in Tulcea (Alt.2)

Line No.	Line No. of	Sewer Length (m)		Max. Flow (m3/s)			Sewer Line				Sewer Invert Elevation (m)		Ground Elevation (m)		Earth Covering (m)		Remark
	Lower Sewer	Increment	Total	Sewage	Infilt.	Total	Diameter (mm)	Slope (o/oo)	V (m/s)	Cap. (m3/s)	Upper end	Lower end	Upper end	Lower end	Upper end	Lower end	
TI0	TI1	390	390	0.040	0.000	0.040	300	1.7	0.667	0.047	3.840	3.180	5.240	4.880	1.100	1.400	*1)
No.1	TI1	-	-	0.002	0.000	0.002	300	-	-	-	3.400	3.400	-	4.900	-	1.200	*1)
TI1	TI2	482	872			0.042	400	2.3	0.939	0.118	3.180	2.070	4.900	4.270	1.320	1.800	*1)
No.2+3	TI2	-	-	0.016	0.000	0.016	300	-	-	-	2.650	2.650	4.450	4.270	1.500	1.320	*1)
TI2	TI3	210	1,082			0.058	400	3.3	1.125	0.141	2.070	1.370	4.270	4.250	1.800	2.480	*1)
No.5-5	TI3	-	-	0.015	0.000	0.015	400	-	-	-	-	2.550	-	4.250	-	1.300	*1)
TI3	TI4	17	1,099			0.073	600	2.4	1.257	0.355	1.370	1.330	4.250	4.030	2.280	2.100	*1)
No.4	TI4	-	-	0.208	0.000	0.208	400	-	-	-	2.860	2.860	4.360	4.030	1.100	0.770	*1)
TI4	TI5	75	1,174			0.281	600	2.6	1.107	0.313	1.330	1.135	4.030	4.550	2.100	2.815	*2)
TI5	SP3	12	1,186			0.281	600	2.6	1.107	0.313	1.135	1.104	4.550	4.500	2.815	2.796	*3)

Remark *1) : Using existing sewer

*2) : Replacement of existing sewer with new sewer

*3) : Newly installation of sewer

Table AII.2.6 Existing Sewer Head in Tulcea

Existing Sewer Head of No.5 (Old Interceptor Sewer)

Point	Calculation				Head loss by friction (m)	Head loss by Other (m)	Total Head loss (m)	Head(H) (m)
Outlet Point (HWL)								5.00
Connection point from the manhole-SP3							0.74	5.74
	Condition							
	Diameter(D)	(mm)	1000					
	Length(L)	(m)	3600					
	Flow (Q), total of No.5	(m3/s)	0.302					
	Velocity(V)	(m/s)	0.385					
	Slope(I)	(o/oo)	0.193		0.69			
	V^2/(2xg)	0.008	Number	Sum(f)				
	f(out)	1	1	1				
	f(sec)	0.5	1	0.5				
	f(bend)	1	5	5				
			Total	6.5		0.05		
Lowest house connection point of Sewer No.5							4.94	10.68
	Condition							
	Diameter(D)	(mm)	400					
	Length(L)	(m)	600					
	Flow (Q), (5-2 and 5-3)	(m3/s)	0.191					
	Velocity(V)	(m/s)	1.52					
	Slope(I)	(o/oo)	7.151		4.29			
	V^2/(2xg)	0.118	Number	Sum(f)				
	f(out)	1	0	0				
	f(sec)	0.5	1	0.5				
	f(bend)	1	5	5				
			Total	5.5		0.65		
Elevation of Lowest house connection point of Sewer No.5								15.00

Existing Sewer Head of No.6 (New Interceptor Sewer)

Point	Calculation				Head loss by friction (m)	Head loss by Other (m)	Total Head loss (m)	Head(H) (m)
Outlet Point (HWL)								5.00
Upper end point of major interceptor(D1000)							0.08	5.08
	Condition							
	Diameter(D)	(mm)	1000					
	Length(L)	(m)	3600					
	Flow (Q), total of No.6	(m3/s)	0.086					
	Velocity(V)	(m/s)	0.109					
	Slope(I)	(o/oo)	0.019		0.07			
	V^2/(2xg)	0.001	Number	Sum(f)				
	f(out)	1	1	1				
	f(sec)	0.5	1	0.5				
	f(bend)	1	5	5				
			Total	6.5		0.01		
Lowest house connection point of Sewer No.6							1.83	6.91
	Condition							
	Diameter(D)	(mm)	350					
	Length(L)	(m)	500					
	Flow (Q), total of No.6	(m3/s)	0.086					
	Velocity(V)	(m/s)	0.894					
	Slope(I)	(o/oo)	3.127		1.56			
	V^2/(2xg)	0.041	Number	Sum(f)				
	f(out)	1	1	1				
	f(sec)	0.5	1	0.5				
	f(bend)	1	5	5				
			Total	6.5		0.27		
Elevation of Lowest house connection point of Sewer No.6								15.20

Table AII.2.7 Planed Sewer Head in Tulcea

Planning Sewer Head after the Project of No.5 (Old Interceptor Sewer)

Point	Calculation				Head loss by friction (m)	Head loss by Other (m)	Total Head loss (m)	Head(H) (m)
Outlet Point (HWL)								5.00
Connection point from the manhole-SP3							0.83	5.83
	Condition							
	Diameter(D)	(mm)	1000					
	Length(L)	(m)	3900					
	Flow (Q), half of total flow to WWTP	(m3/s)	0.307					
	Velocity(V)	(m/s)	0.391					
	Slope(I)	(o/oo)	0.199		0.78			
	V^2/(2xg)	0.008	Number	Sum(f)				
	f(out)	1	1	1				
	f(sec)	0.5	1	0.5				
	f(bend)	1	5	5				
			Total	6.5		0.05		
Manhole from the SP3							3.96	9.79
	Condition							
	Diameter(D)	(mm)	400					
	Length(L)	(m)	175					
	Flow (Q), SP-3	(m3/s)	0.281					
	Velocity(V)	(m/s)	2.236					
	Slope(I)	(o/oo)	14.615		2.56			
	V^2/(2xg)	0.255	Number	Sum(f)				
	f(out)	1	0	0				
	f(sec)	0.5	1	0.5				
	f(bend)	1	5	5				
			Total	5.5		1.40		
Elevation of the manhole								15.00
Lowest house connection point of Sewer No.5							4.94	10.77
	Condition							
	Diameter(D)	(mm)	400					
	Length(L)	(m)	600					
	Flow (Q), (5-2 and 5-3)	(m3/s)	0.191					
	Velocity(V)	(m/s)	1.52					
	Slope(I)	(o/oo)	7.151		4.29			
	V^2/(2xg)	0.118	Number	Sum(f)				
	f(out)	1	0	0				
	f(sec)	0.5	1	0.5				
	f(bend)	1	5	5				
			Total	5.5		0.65		
Elevation of Lowest house connection point of Sewer No.5								15.00

Planning Sewer Head after the Project of No.6 (New Interceptor Sewer)

Point	Calculation				Head loss by friction (m)	Head loss by Other (m)	Total Head loss (m)	Head(H) (m)
Outlet Point (HWL)								5.00
Upper end point of major interceptor(D1000)							0.83	5.83
	Condition							
	Diameter(D)	(mm)	1000					
	Length(L)	(m)	3900					
	Flow (Q), half of total flow to WWTP	(m3/s)	0.307					
	Velocity(V)	(m/s)	0.391					
	Slope(I)	(o/oo)	0.199		0.78			
	V^2/(2xg)	0.008	Number	Sum(f)				
	f(out)	1	1	1				
	f(sec)	0.5	1	0.5				
	f(bend)	1	5	5				
			Total	6.5		0.05		
Lowest house connection point of Sewer No.6							1.83	7.66
	Condition							
	Diameter(D)	(mm)	350					
	Length(L)	(m)	500					
	Flow (Q), half of total flow to WWTP	(m3/s)	0.086					
	Velocity(V)	(m/s)	0.894					
	Slope(I)	(o/oo)	3.127		1.56			
	V^2/(2xg)	0.041	Number	Sum(f)				
	f(out)	1	1	1				
	f(sec)	0.5	1	0.5				
	f(bend)	1	5	5				
			Total	6.5		0.27		
Elevation of Lowest house connection point of Sewer No.6								15-20

Table AII.2.8 Quantity of Planned Interceptor System in Tulcea

Connection Sewer

Diameter (mm)	Earth Covering (m)				Total Length (m)	Remark
	1-3	3-5	5-7	7-9		
200	10				10	Connection sewer for Outfall No.1 to interceptor
200	10				10	Connection sewer for Outfall No.2 to interceptor
200	10				10	Connection sewer for Outfall No.3 to interceptor It is new construction but site is conjusted.
400	20				20	Connection sewer for Outfall No.4 to interceptor

Interceptor Sewer

Diameter (mm)	Earth Covering (m)				Total Length (m)	Remark
	1-3	3-5	5-7	7-9		
400	173				173	from receiving manhole to major interceptor
600	75				75	Replacement of sewer Line No. TI4
600	12				12	New installation of sewer Line No. TI5
1000	10				10	Connection sewer between major interceptor near S-SP3
1000	590				590	Diversion of major interceptor to WWTP 2 x 295 m

Pressure Sewer

Diameter (mm)	Earth Covering (m)				Total Length (m)	Remark
	1-2					
400	285				285	From S-SP3 to receiving manhole Steel pipe

Manhole and Receiving Tank

Diameter (mm)	Earth Covering (m)				Total No. of (pcs)	Remark
	1-3	3-5	5-7	7-9		
500	1				1	at Outfall No.4 to interceptor
600	1				1	at junction from outfall No.4
Receiving Tank	1				1	Receiving tank from S-SP3 (Dia3mxdepth3m)

Valve

Diameter (mm)	Earth Covering (m)				Total No. of (pcs)	Remark
	1-3	3-5	5-7	7-9		
1000	2				2	near junction from receiving tank
1000	3				3	near WWTP
Total	5				5	

Pump equimpment at S-SP3

Q	0.15	m3/s
H	30	m
Number	3 (1)	pcs One pump is standby.

Pump equipment will be installed at the existing stormwater pumping (S-SP3) structure.

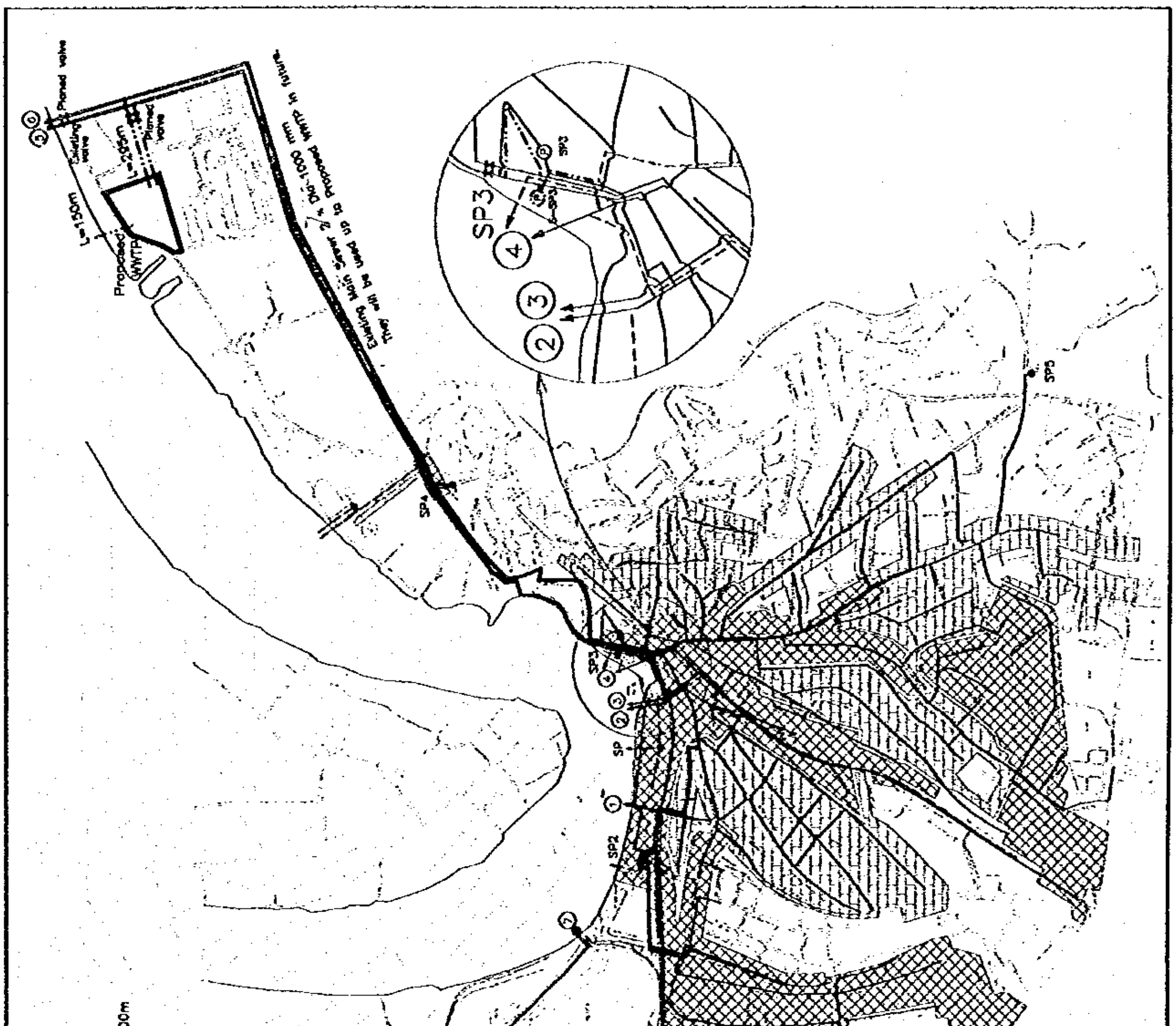


Figure AII.2.1	Tulcea City Sewerage System (Existing and Planning)

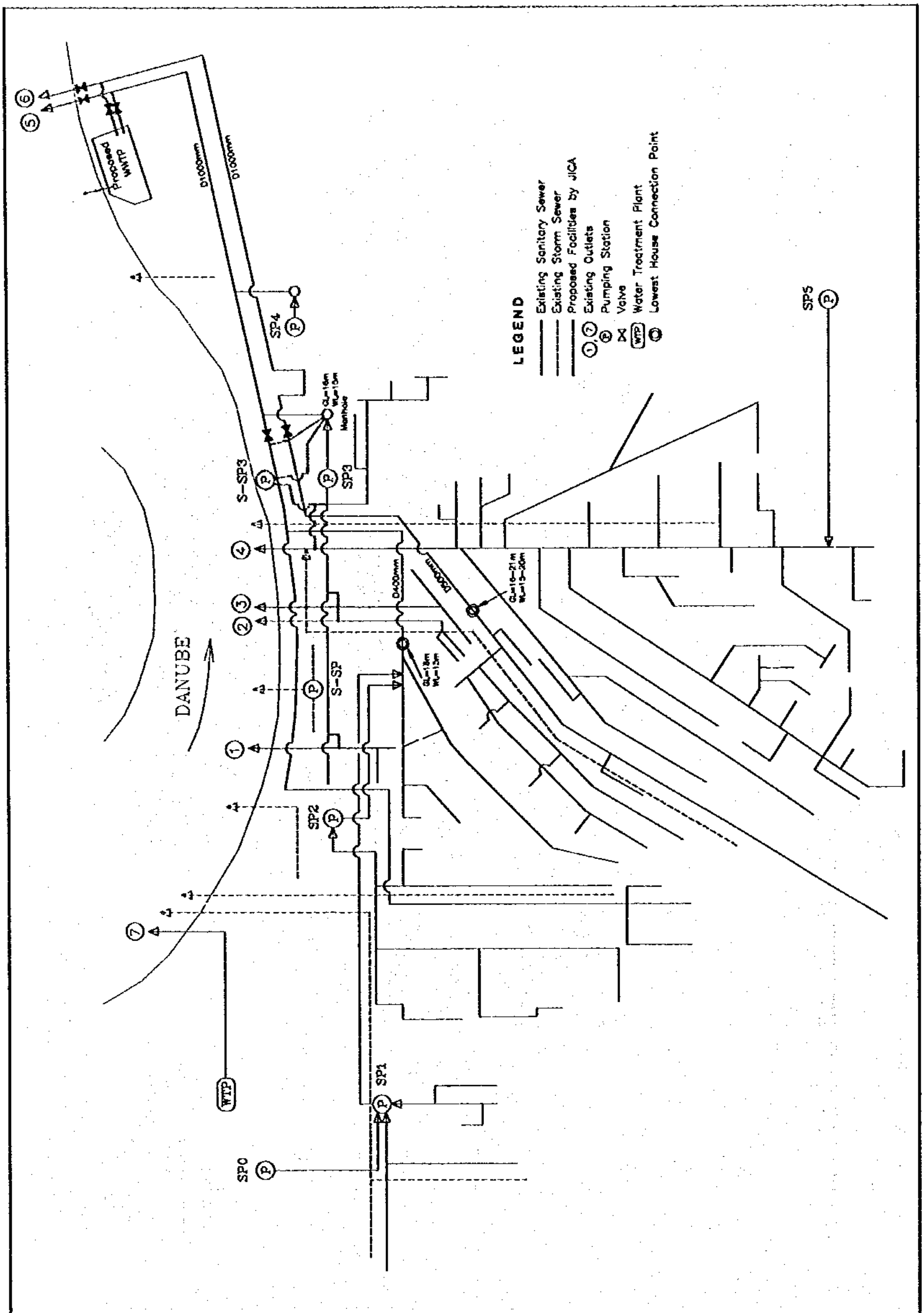


Figure AII.2.2	Schematic Representation of Tulcea Sewerage System (Existing and Planning)

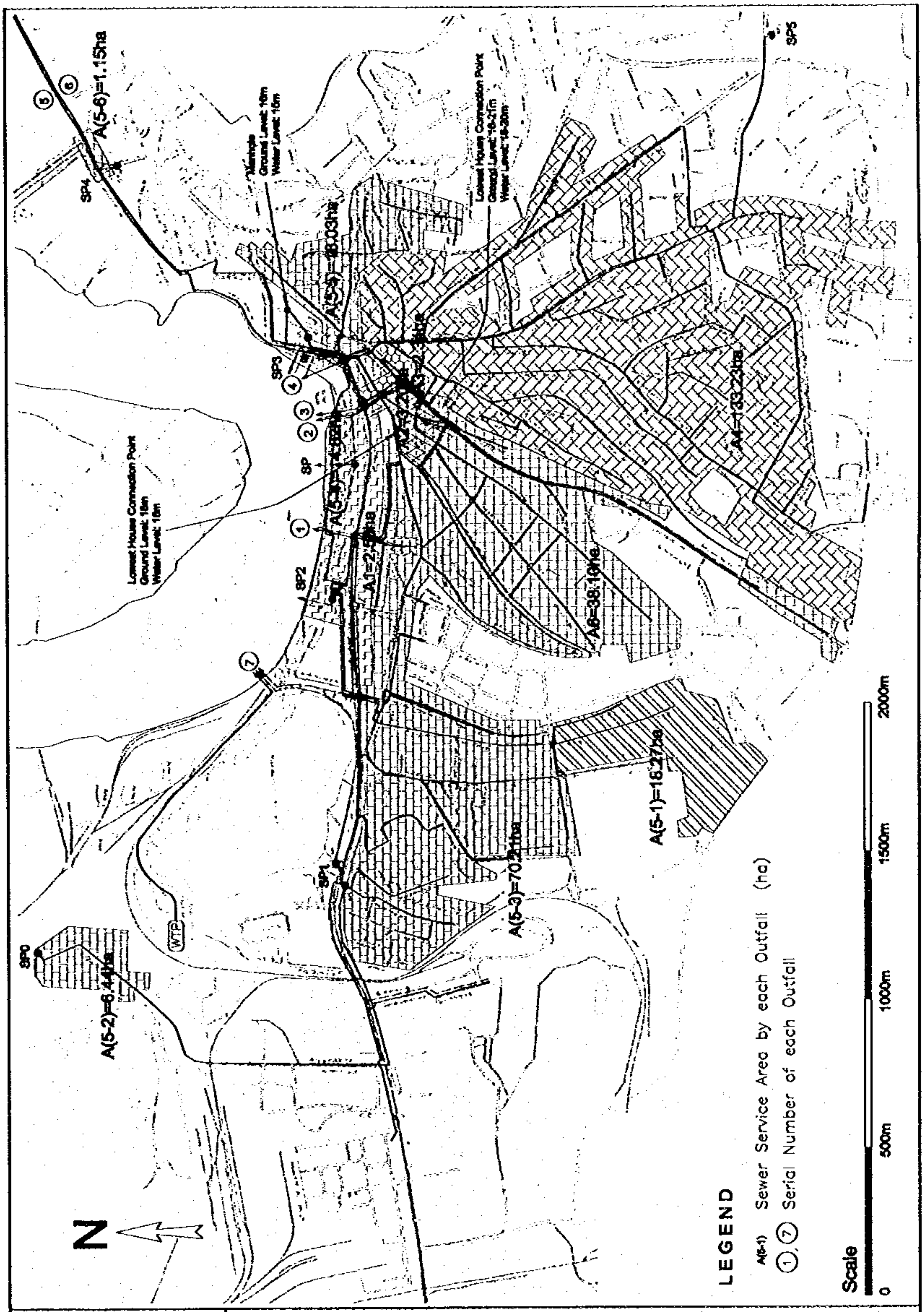


Figure AII.2.3	Existing Sewer Service Area by each Outfall

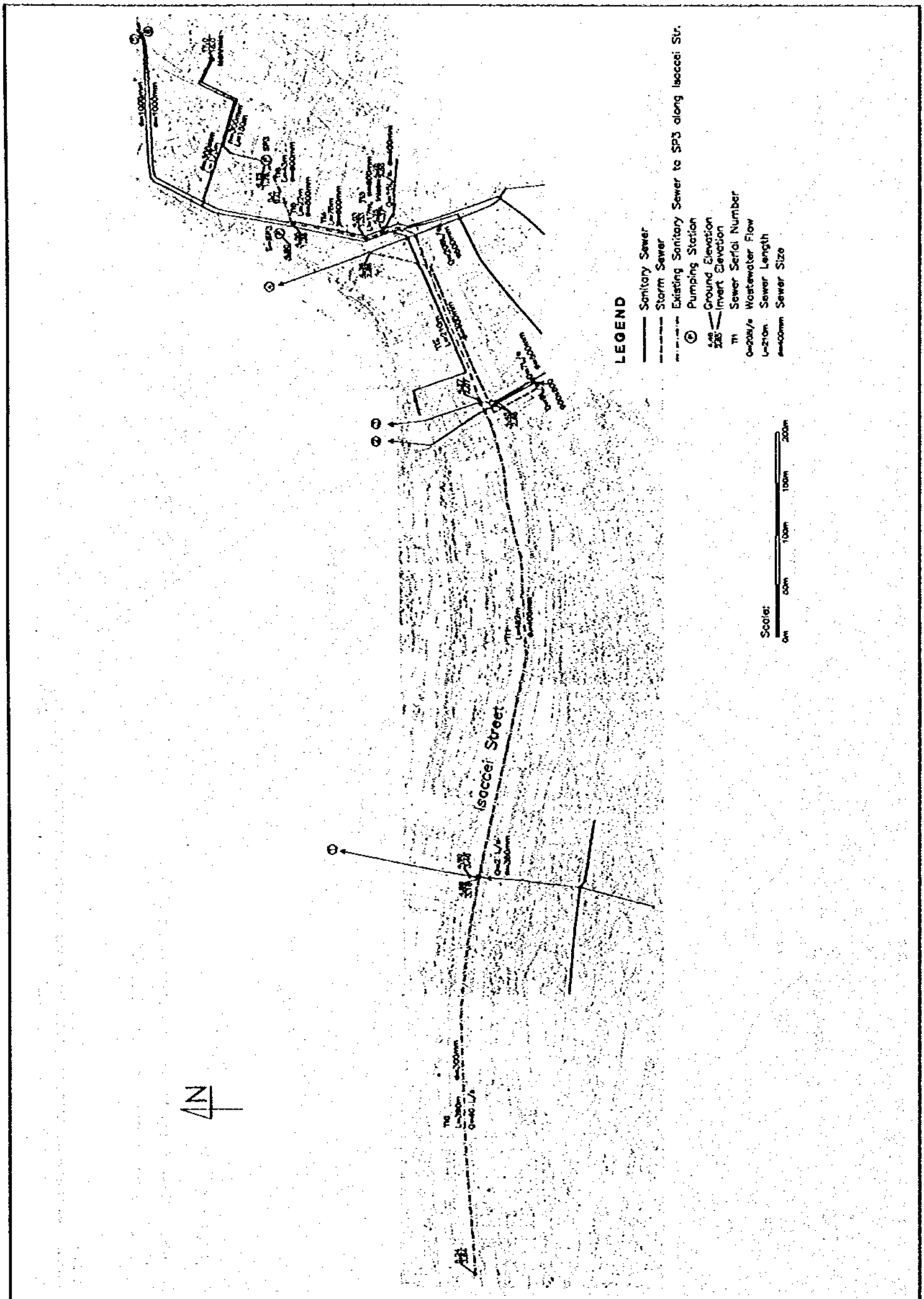
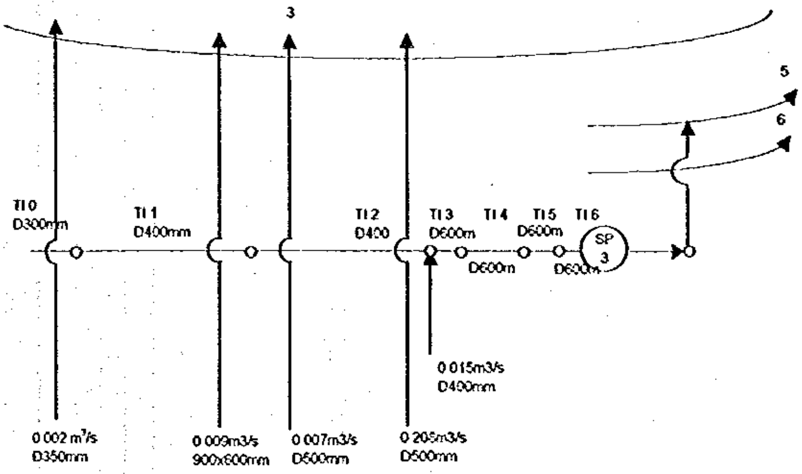
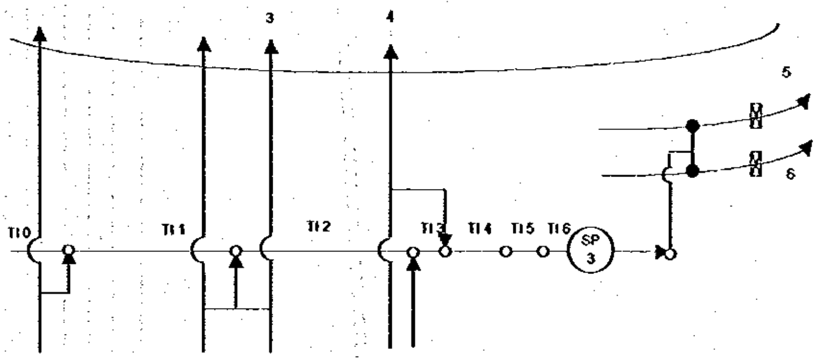


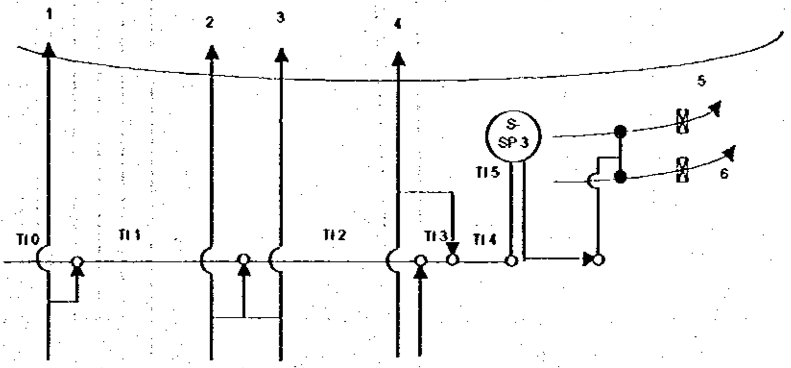
Figure AII.2.4	Existing Sewer Layout around Planned Interceptor in Tulcea



Existing Sewer Layout



Interceptor Plan (Alt 1)



Interceptor Plan (Alt 2)

Figure AII.2.5	Flow Chart of Existing Sewer and Planned Interceptor of Tulcea

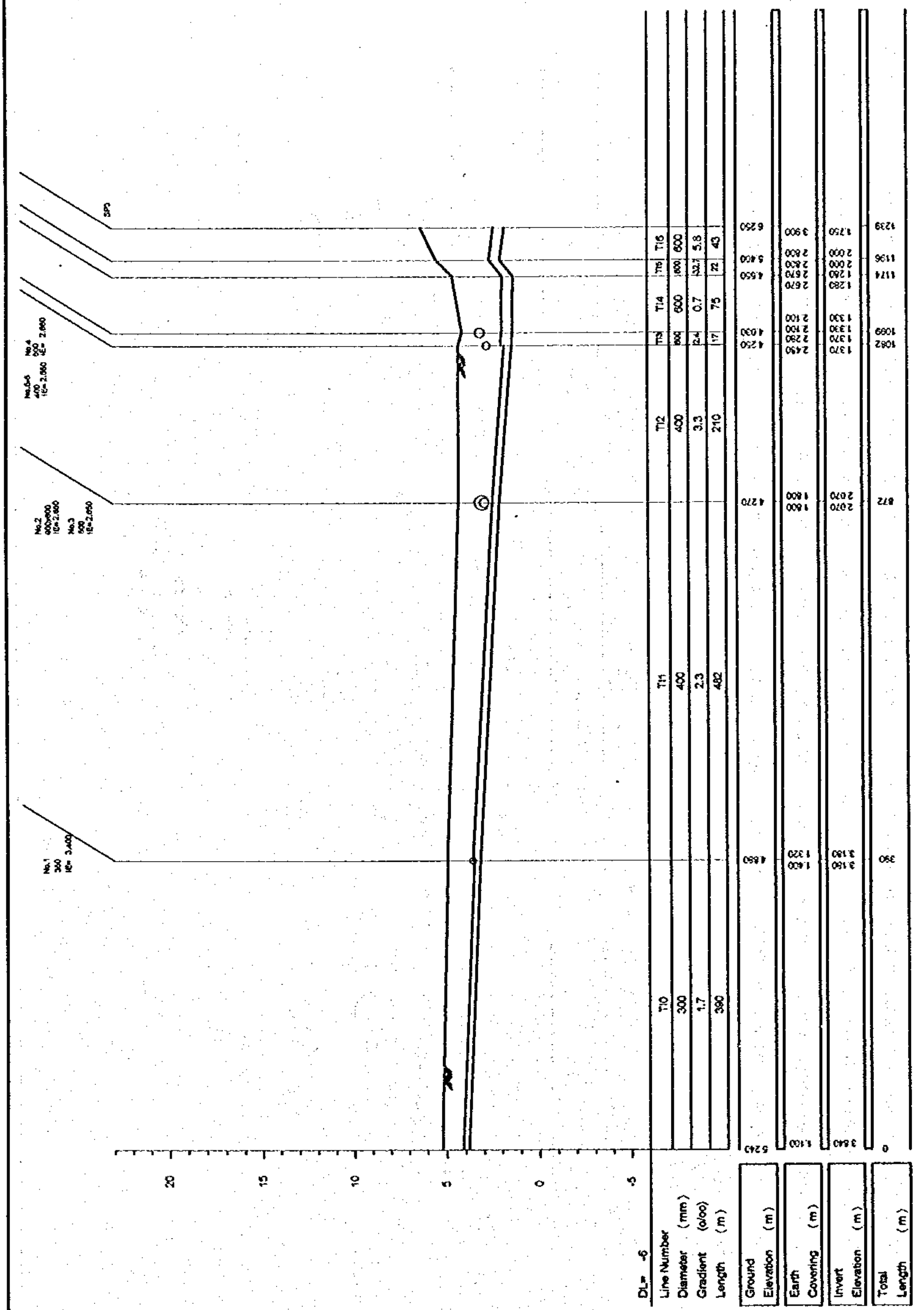


Figure AII.2.6	Longitudinal Profile of Existing Sewer to SP3 in Tulcea

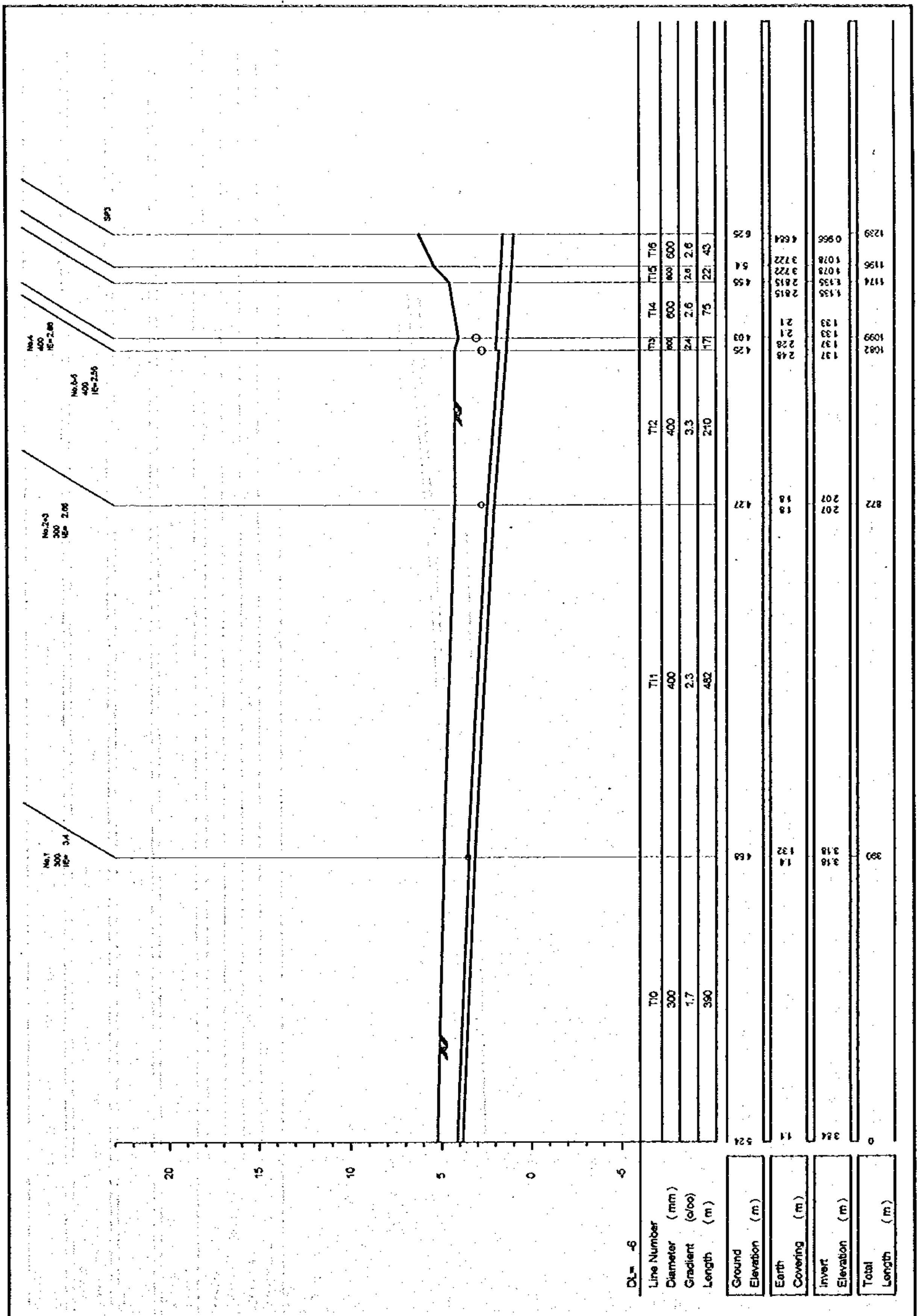


Figure AII.2.7	Longitudinal Profile of Planned Interceptor to SP3 in Tulcea (Alt.1)

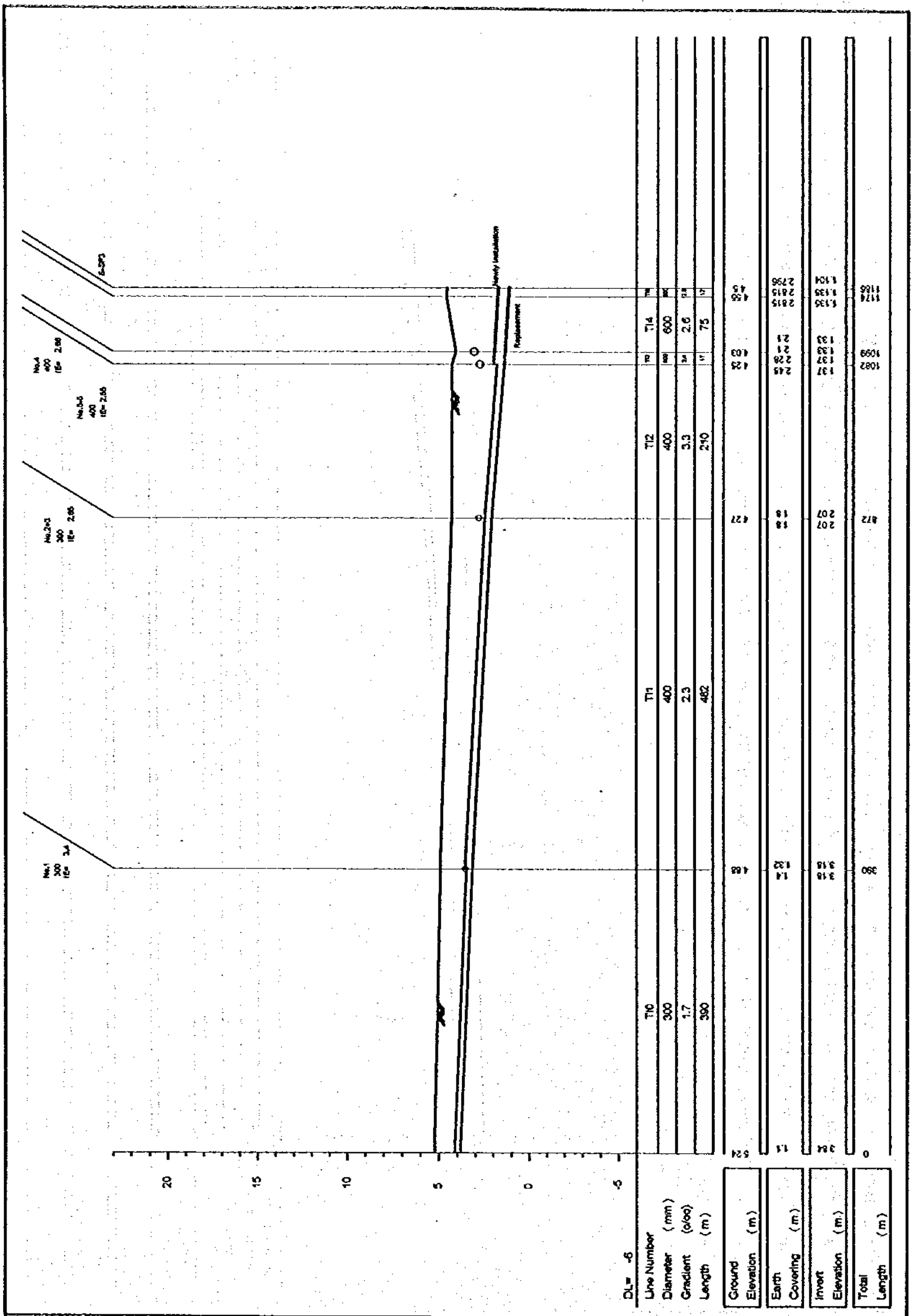


Figure AII.2.8	Longitudinal Profile of Planned Interceptor to S-SP3 in Tulcea (Alt.2)

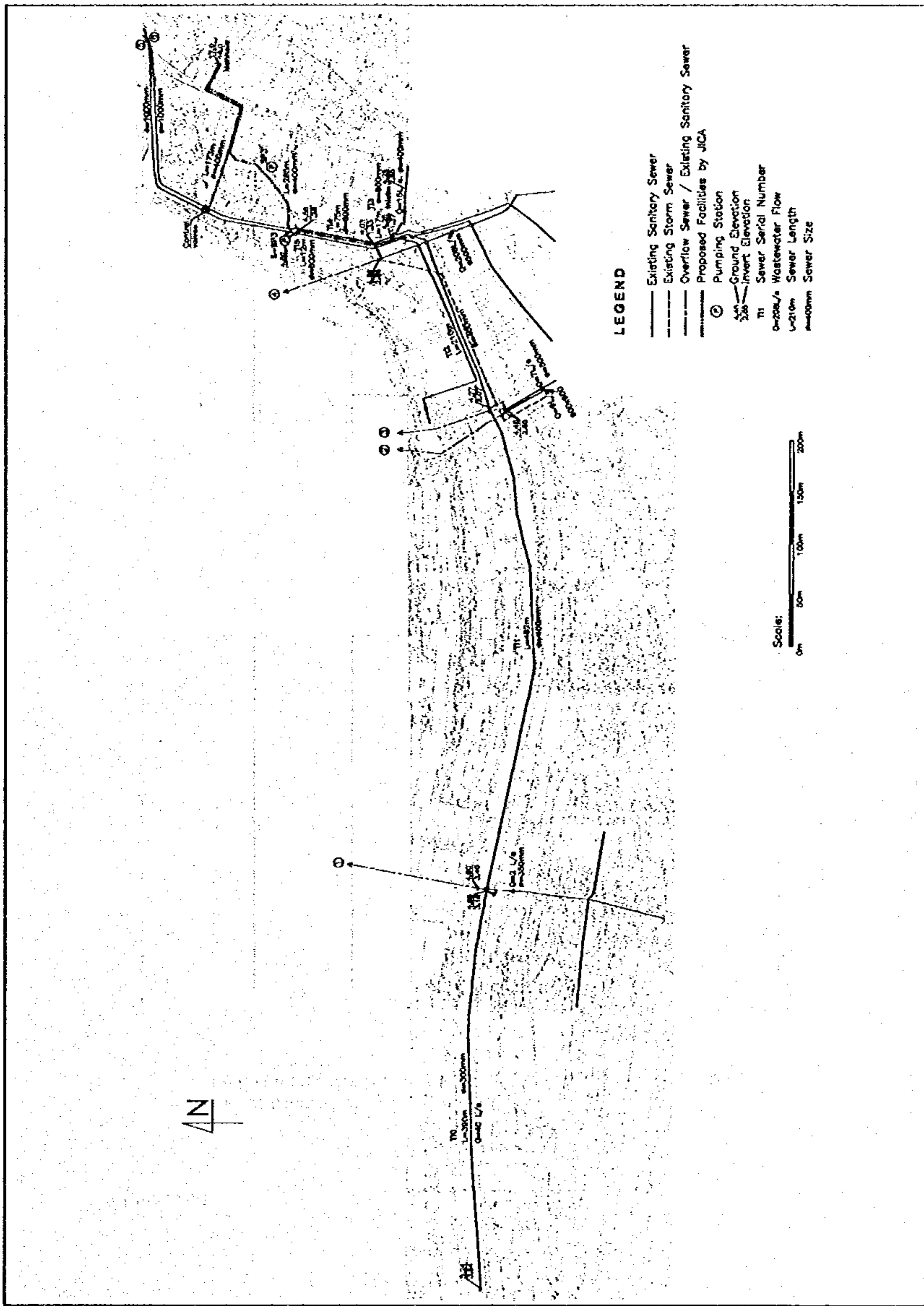
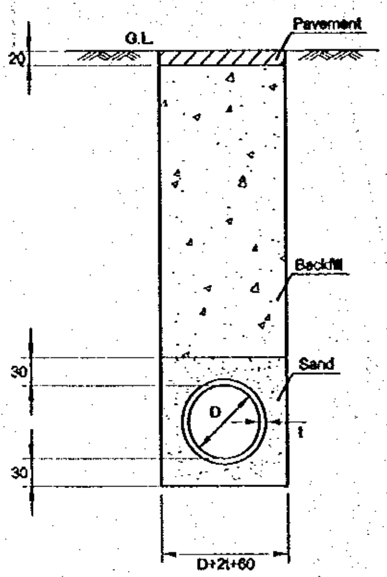
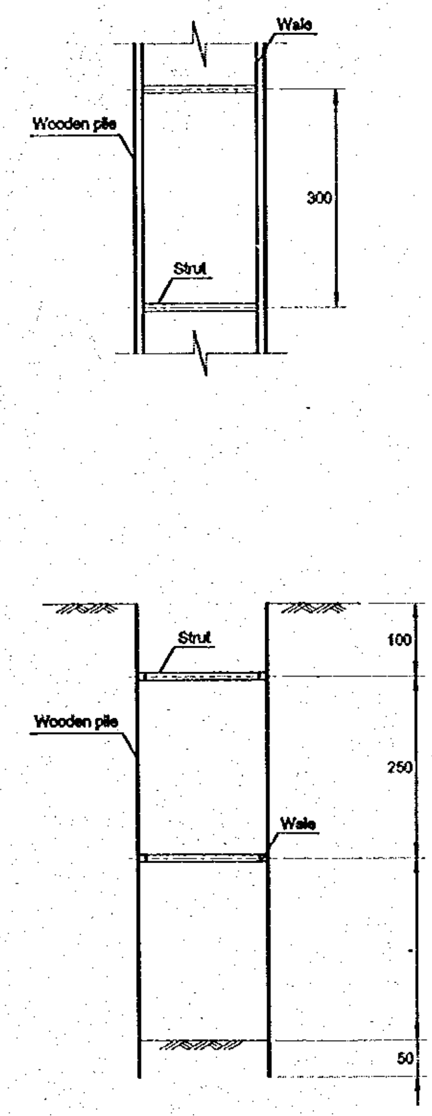


Figure AII.2.9	Layout of Planned Interceptor in Tulcea (Alt.2)



Typical Sewer Bedding



Typical Sheathing on
Sewer Construction

(unit : cm)

Figure AII.2.10	Typical Sewer Bedding and Typical Sheathing on Sewer Construction

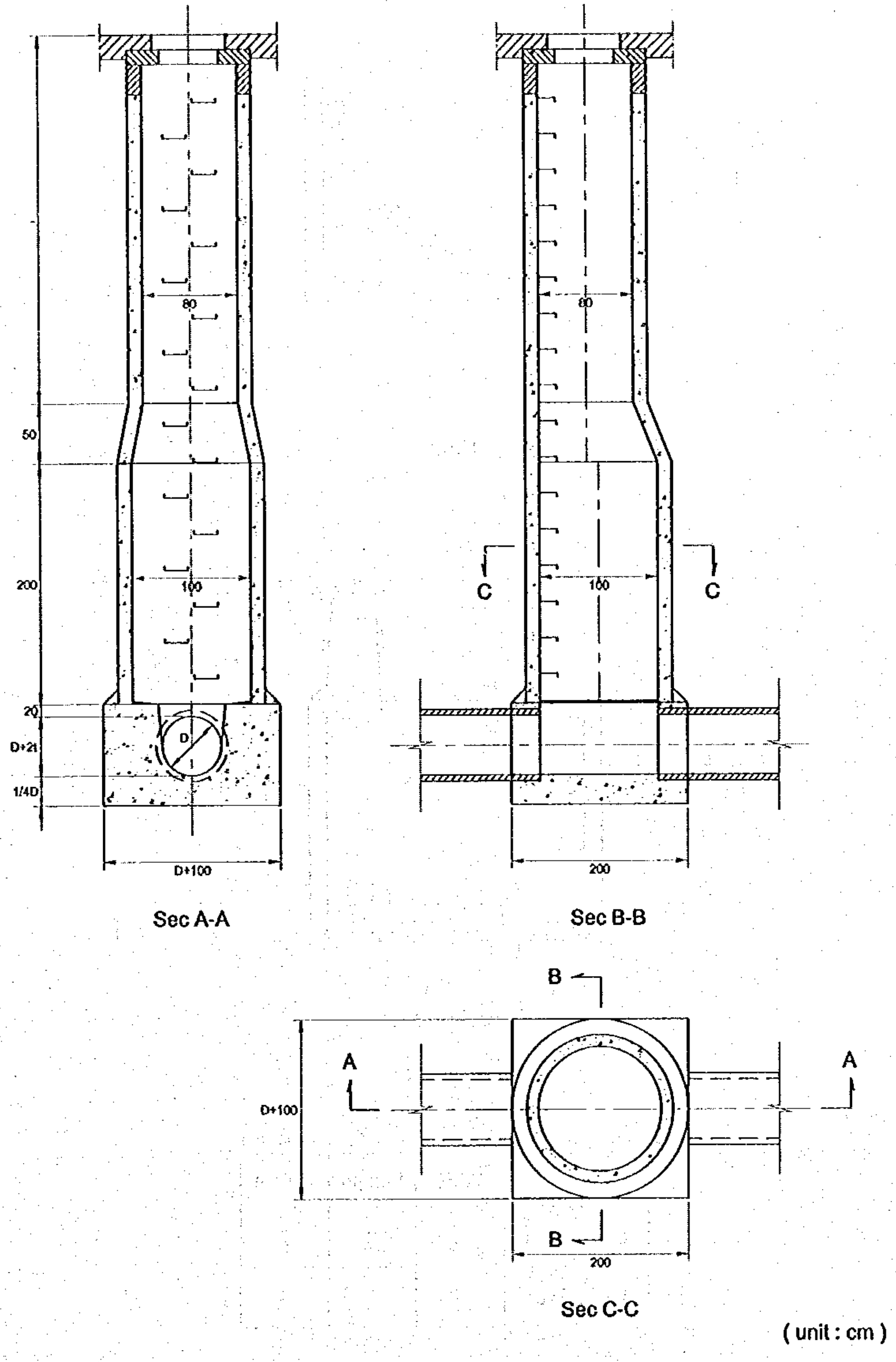
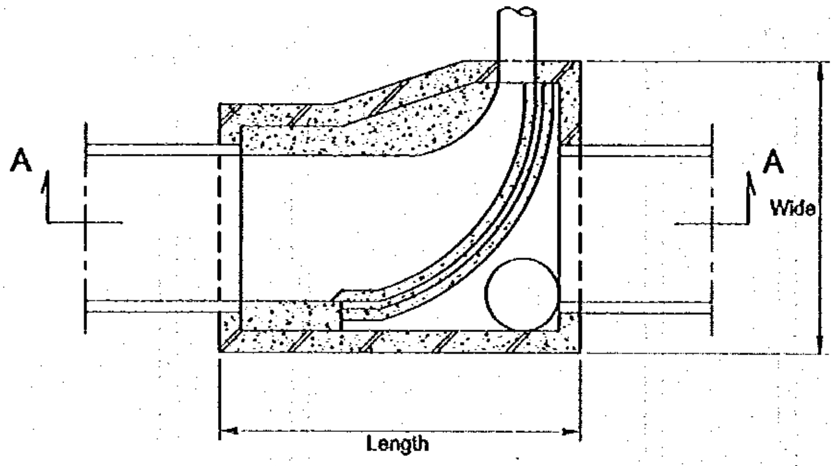
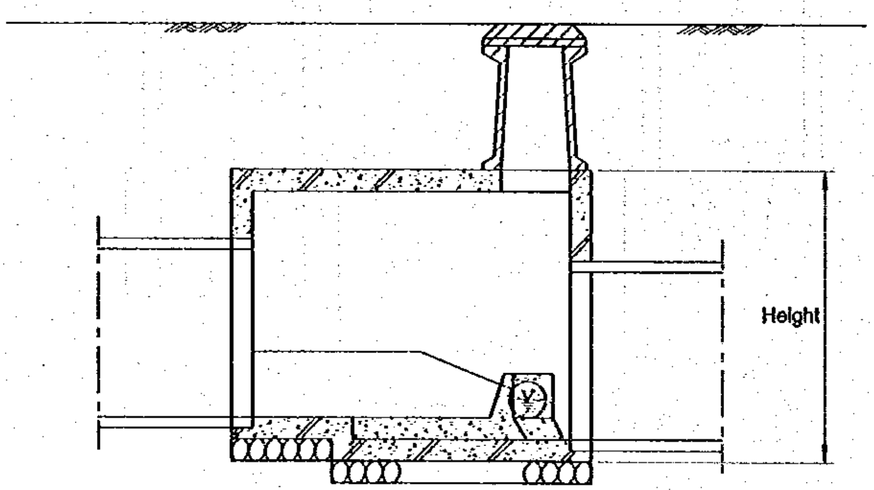


Figure AII.2.11	Typical Stucture of Manhole



Plan



Section A-A

(unit : m)

	Wide	Length	Height
Small Type	2	3	2
Large Type	4	5	4

Figure AII.2.12	Typical Combined Sewer Overflow (CSO) Regulator

APPENDIX-3 FACILITY PLANNING

1. PLANNING PRINCIPLE

1.1 HYDRAULIC/ORGANIC LOADING ON FACILITIES

In determining the hydraulic and organic capacities of the WWTP component facility, the maximum daily wastewater flow of 43,000 m^3/day or 0.498 m^3/sec is employed. Each component facility is further checked both for the average daily flow and maximum hourly flow conditions. No over-topping of any structure under any condition is considered.

All piping and channels are designed to carry the maximum hourly flow rate. For the design purpose, the inflowing wastewater through the existing pressure sewer pipes (2 x 1,000 mm in diameter) is considered to enter the influent chamber ahead of the WWTP by gravity.

1.2 FLOW DIVISION CONTROL

The plant facility design provides for flow division control facilities to insure organic and hydraulic loading control to various process units. Convenient, easy and safe access, change, observation, and maintenance are considered in the design of such facilities. Flow division will be measured using flow measurement devices to assure uniform loading of all unit processes and operations.

1.3 UNIT BYPASSES

A minimum of two units in the liquid treatment process train shall be provided for all unit processes and operations in the plant. The design will provide for properly located and arranged bypass structures and piping so that each unit of the plant can be removed from service independently. The bypass design will facilitate plant operation during unit maintenance and emergency repair so as to minimize deterioration of effluent quality and insure rapid process recovery upon return to normal operational mode.

1.4 PIPE CLEANING AND MAINTENANCE

Fittings, valves, and other appurtenances should be provided for pipes subject to clogging, to facilitate proper cleaning through mechanical cleaning or flushing. Pipes subject to clogging, such as pipes carrying sludge, are to be lined with a material which creates a smooth and non-adhering surface, thereby reducing clogging and resistance to flow.

1.5 CONSTRUCTION MATERIALS

The materials of construction and equipment shall be resistant to hydrogen sulfide and other corrosive gases, greases, oils, chemicals, and similar constituents frequently present in sewage. This is particularly important in the selection of metals and paints. Contact between dissimilar metals should be avoided to minimize galvanic action, and consequent corrosion.

1.6 GRADING AND LANDSCAPING

The plant site should be graded and landscaped upon completion of the plant. Concrete or asphalt paved walkways should be provided for access to all units. Steep slopes should be avoided to prevent erosion.

1.7 PLANT OUTFALL LINES

The Tulcea WWTP outfall sewers are located and designed to discharge the treated wastewater effluent to the Danube River through in a manner not to impair the beneficial uses of the receiving stream, providing for:

- Free fall or submerged discharge at the site; and
- Limited or complete dispersion of discharge across stream to minimize impact on aquatic life movement, and growth in the immediate reaches of the receiving stream.

The outfall structures should be so protected against the effects of floodwater, ice, or other hazards as to reasonably insure its structural stability and freedom from stoppage. The outfall lines should have a safe and convenient access, preferably using a manhole, so that a sample of the effluent can be obtained at a point after the final treatment process, and before discharge to or mixing with the receiving waters.

1.8 SITE DEVELOPMENT

1.8.1 SELECTED SITE

The City-owned WWTP site of about 5.1 hectares land is located at the right bank of the Danube River. The land is relatively flat and low-lying with the ground surface elevation ranging from 2.5 m to 3.5 m above the M.W.L. This land area can accommodate the activated sludge WWTP component facilities to treat the maximum daily wastewater flow of 43,000 m^3/day.

In the vicinity of the plant site are at present agricultural and industrial areas, which conditions may remain unchanged in the foreseeable future. There exists several residences within a distance of 300 m from the boundary of the site, but to the east, there is a wide Government-owned vacant land that could be used for the future plant expansion.

1.8.2 Ground Preparation

The site is an abandoned industrial land with sandy surface soil having a relatively low permeability. The land is flat with ground elevations ranging from the highest point of +3.5m to the lowest point of +2.5 m M.W.L. There will be some differences in the ground elevations between the surrounding natural grounds and the site after the land is prepared.

Because of the ground conditions, the plant site should be developed on account of the following factors:

- Prepare a flat ground surface for economical and technical reasons;
- Prevent possible malfunction of the plant operation due to the flooding;
- Provide easy access to the plant;
- Reduce the cost for earth works, e.g. excavation and backfilling; and
- Reduce pumping head requirements within the treatment plant system.

The treatment plant structures and all related equipment shall be protected from physical damage preferably by the 100-year flood. According to the Danube River hydrological data recorded at a location near the WWTP site over the last 28 years (1970 to 1998), the highest water elevation was 4.35 m above M.W.L. that occurred on 25th May, 1970. But no data are available to indicate the 100-year flood elevation. The 100-year flood elevation of the River is therefore calculated to be + 4.70 m M.W.L. by using the available hydrological data.

The River Bank elevation, at the WWTP effluent outfall location is 5.84 m above M.W.L. whereas the land surface elevation ranges from +2.5m to +3.5m M.W.L. On account of the conditions, the finished ground surface elevation of the site shall be at least + 4.7 m M.W.L. or higher. As the present average ground elevation of the site is +3.0 m M.W.L., a landfill of about 1.7 m will be required.

1.8.3 Site Access

Access to the site can be made through a major road, running from west to east along the Danube River. From the major road, an existing unpaved 6 m wide public road can be used as the access road to the site.

1.8.4 Drainage

The ground surface will be finished to the level of +4.7 m M.W.L. that is not subject to flooding. The only inundation within the site might be of stagnated stormwater. Hence, a drainage facility should be provided to handle the stormwater runoffs. The drainage may be of open channels or conduits installed along road edges and then discharge the stormwater into the nearby drains.

1.8.5 Water Table/Soil Profiles

The groundwater table in the site is relatively high, and may affect to the deep underground structures. Soil bores were prepared at the site.

2. ESSENTIAL FACILITIES

2.1 EMERGENCY POWER FACILITIES

The plant shall have an alternate source of electric or mechanical power to allow continuity of operation during power failures, including provision of at least two independent sources of power, such as feeders, grid, etc., to the plant, or power generators.

Although standby power generating capacity normally is not required for aeration equipment used in the activated sludge type processes, auxiliary power for minimum aeration of the activated sludge is required to protect downstream uses.

2.2 PLANT SANITARY SYSTEM

2.2.1 Water Supply System

An adequate supply of potable water under pressure shall be provided for use in the laboratory and for general cleanliness around the plant. No piping or other connections shall exist in any part of the treatment works which, under any conditions, might cause the contamination of the potable water supply.

Potable water from a municipal or separate supply will be used directly at points above grade for hot and cold supplies in lavatory, water closet, laboratory sink (with vacuum breaker), shower, drinking fountain, eye wash fountain, and safety shower; unless local authorities require a positive break at the property line.

Hot water for any of the above units shall not be taken directly from a boiler or piping used for supplying hot water to a sludge heat exchanger or digester heating unit.

Where a potable water supply is used for any purpose in a plant, a break tank, pressure pump, and pressure tank shall be provided. Water shall be discharged to the break tank through an air gap at least 15 cm above the maximum flood line or the spill line of the tank, whichever is higher.

2.2.2 Other Sanitary Facilities

Toilet, shower, lavatory, and locker facilities shall be provided in convenient locations to serve the expected staffing level at the plant.

2.3 Flow Measurement

A Parshall flume should be provided after the preliminary treatment facility to continuously indicate, totalize and record the volume of the wastewater entering the plant in a unit time. Locations close to turbulent, surging or unbalanced flow, or a poorly distributed velocity pattern should be avoided. The Parshall flume should be provided only in locations where free discharge conditions exists on the downstream side at the average design flow.

2.4 Plant Bypass

The WWTP design calls for accepting the peak flow of 0.614 m^3/sec. Flows in excess of this rate may be bypassed to the nearby waterways or the Danube River. In the flow bypass structure a broad-crested weir should be set at a calculated hydraulic grade line elevation, which will accomplish this maximum hydraulic plant loading limitation.

The plant bypass should be constructed at the location ahead of the WWTP. As much portion of the wastewater will be sent to the WWTP through wastewater lift pumping stations, the frequency of the flow exceeding 0.614 m^3/sec. is expected to be extremely low.

2.5 Laboratory

The WWTP shall include a laboratory for making the necessary analytical determinations and operating control tests. The laboratory size, bench space, equipment and supplies shall be such that it can perform analytical work for:

- All self-monitoring parameters required by discharge permits;
- The process control necessary for good management of each treatment process included in the design; and
- Industrial waste control or pretreatment programs.

3. Process Design

This section describes the design basis for the required wastewater treatment process components, which have to be developed as a preliminary step to the detailed design.

3.1 Hydraulic Loading of Process Facilities

The hydraulic loading rates used for the capacity design of process components are summarized as follows:

Design Hydraulic Loads of Plant Component Facilities

	Component Facilities	Hydraulic Loads (m^3/sec)	
		Design hydraulic load (Maximum daily flow)	Maximum hourly flow
1	Preliminary Treatment	0.498	0.614
2	Influent pumping station	0.498	0.614
3	Primary Treatment	0.498	0.614
4	Secondary Treatment	0.498	0.614
5	Chlorination tank	0.498	0.614
6	Effluent pumping station	0.498	0.614
7	Sludge management	0.498	0.614
8	All pipes and conduits	-	0.614

Note: All the facilities can accommodate the maximum hourly flow.

The hydraulic capacities of the component facilities are determined on the basis of the maximum daily flow rate and checked for the conditions of average daily and maximum hourly flows, to ascertain that all the facilities can accommodate the maximum hourly flow rate without hydraulic hindrance.

3.2 PRELIMINARY TREATMENT

The process units and structures associated with the preliminary treatment are the influent gates, screens (coarse/ fine), aerated grit removal, flow measurement, and influent pumping.

3.2.1 Influent Gates

Gates: At the entrance to the plant, manually operated influent gates are provided to control or bypass the influent flows. The design data of the gates are as follows:

Number of gates	2 units
Type	cast-iron made, sluice gate (manually operated)
Gate size	1.2 m x 1.2 m
Maximum head loss	about 100 mm

3.2.2 Screens

In view of the necessity for the efficient operation of the rakes and also for reducing the hydraulic head losses in the screens, the wastewater flow velocity reaching at the channels is planned to be at around 1.0 m/sec.

Coarse Screens: Manually-cleaned coarse bar screens are provided ahead of the fine screens. The criteria for the coarse screens are as follows:

Number of screens	2 units
Channel width	1.6 m
Clear bar spacing	100 mm
Slope from vertical	60 degrees

Fine Screens: All the wastewater inflows require fine screening. The fine screens are mechanically cleaned. The criteria for the fine screens will be as follows:

Number of screens	2 units
Channel width	1.6 m

Clear bar spacing	20 mm
Slope from vertical	75 degrees

Arrangement of Screening Facility: The number of individual screening units should be such that when one unit is taken out of service for maintenance or repair, the remaining units can accommodate the additional screening load with ease. Captured screenings should be kept in closed containers until the screenings are transported to the disposal site for sanitary landfill.

Attention must be paid in the design so that drainage from the screenings is not spilled on the floors. Facilities for down washing the equipment and storage areas must also be provided. This includes appropriate floor drains and piping to return the wastewater back to the head of the plant.

Screenings Disposal: The screenings will contain organic and putrescible materials, and if not disposed of quickly, will represent an attraction and breeding ground for flies and other insects. As a minimum, screenings must be disposed of daily. All collected screenings will be dumped to one common belt conveyor and sent to a hopper for storage, then, dumped into a truck for hauling it to a sanitary landfill.

Controls: The electric motors that activate the raking mechanism of the bar screens are operated either by manual on-off switch or automatically by clock-operated timing switches.

The clock-operated timers will be set on the basis of experience, which has provided a record of the number of raking operations required during an average day. These timers are set to operate the raking mechanism for the number of times required per day.

Degree of Instrumentation: The degree of instrumentation consists of providing clock-operated timing switches.

3.2.3 Aerated Grit Removal

Type of Grit Chamber: Grit settled by aeration at the bottom of the grit chambers is removed by a grit lifting pump with trolley to a grit channel. The grit in the channel will be sent with a screw conveyor to a grit hopper or bin for final disposal.

Configuration: The grit removal of the wastewater will be accomplished in 2 grit chambers. The geometry of chambers is as follows:

Number of units	2 chambers
Width	3 m (including oil separator)
Length	16 m
Depth	2.35 (side depth) to 3.05 m
Blowers	3 units (one-standby) 80 mm dia. x 5 m^3/min.
Influent gates	2 units, 800mm x 800 mm
Effluent gates	2 units, 600mm x 600 mm
Grit lifting device	2 units of trolley with grit removing pump
Grit lift pumps	2 sets of sand pumps
Screw conveyor	1 unit
Grit hopper	1 unit

Air Supply: For the total tank length of 16 m, air supply requirement is 10 m^3/min. Three units (one-standby) of turbo blowers, each with air supply rate of 5 m^3/min., will be provided.

Grit Removal: A trolley with grit lifting device for grit removal from each chamber as well as one grit pump for each chamber will be operated manually. The grit water pumps lift the grit mixed with water to the grit separator for grit removal. The removed grit will be conveyed to a hopper, which will dump the grit into trucks for hauling to a sanitary landfill. The separated wastewater should be returned to the head of the plant for further treatment.

Controls: Control of the air supply to the grit chambers will be made through valves on the air down-comer pipes leading into each chamber. The air flows to each down-comer will be set manually by the valve which is followed with a mechanical air flow meter to indicate and control equal air quantity to each chamber.

Degree of Instrumentation: The total air quantity supplied to the chambers will be indicated and transmitted to the main central room.

Data Logging: The data to be logged will be the total air flow supplied to the chambers.

Operational: The only operational control of air quantity supplied to each chamber is by setting the mechanical valves to a flow quantity rate indicated on the downstream mechanical orifice type flow meter. The total air quantity provided by the blowers to the chambers will be indicated and transmitted automatically to the main control room through an electrical signal attached to the orifice flow meter located downstream from the blowers.

3.2.4 Flow Measurement

Because of a simplicity of function and ease of operation, a Parshal flume (7 feet) will be installed for measurement of flows after having passed through the grit chambers.

Degree of instrumentation: The degree of instrumentation for the flow meter should be minimal. Conversion of water flow to a flow rate signal is required.

Data Logging: Records of flow rate will be logged.

3.3 PRIMARY TREATMENT

The primary treatment system consists of gravity liquid/solid separation in circular clarifiers. The clarifier system will consist of clarifier modules of 4 units.

3.3.1 Flow Distribution

Following the Parshall flume, the wastewater flows down through the conduit to the distribution chamber located at the center of the cluster of 4 primary clarifiers, from where the flow will be distributed to each individual clarifier. The flow split is proportional to the tank surface area.

3.3.2 Primary Clarifiers

Hydraulic Loading and Area Requirements: The hydraulic loading rate for the clarifiers is 25 $m^3/m^2/day$ at the maximum daily flow of 0.498 m^3/sec. Thus, the total required surface area is calculated to be 1,692 m^2. Based on these data, 4 primary clarifirs are required. The primary clarifier design criteria are summarized as follows:

Surface loading (at maximum daily flow)	35 $m^3/m^2/day$
Design flow rate	43,000 m^3/day
Surface area of each clarifier	423 m^2
Clarifier diameter	35 m

Effective water depth	2.0 m
Number of clarifiers	4 basins
Number of clarifier cluster	1 cluster

Primary Sludge Production: The primary and excess sludge production (when excess sludge is returned to the primary tanks) for the daily average flow rate is 402 m^3/day or 0.28 m^3/min. at an average solids concentration of 2 %. For the maximum process and operational flexibility, the design enables biological excess (waste) sludge withdrawal handling is either accomplished directly from the secondary clarifiers or from the primary clarifiers.

Sludge Pumping to Digestion Facilities: The sludge pumping cycle is based on a sludge blanket measurement. The quantities of sludge, primary plus excess sludge from the flow of 43,000 m^3/day have been calculated as shown below:

Sludge volume	402 m^3/day
TSS	8,040 kg/day
Solids concentration	2.0 %

Scum Management: Scum is removed from the clarifier surface by a rotating scum removal mechanism to a scum pit located near the tanks and is pumped from there to a scum drum screen for scum removal. The filtrate is then returned to the plant inlet channel for further treatment. All removed scum and oil will be disposed of at the sanitary landfill. A single, common sump serves all the sedimentation tanks. The frequency of scum pumping will be determined on the basis of the scum pit level control.

Controls: The clarifier operation will be manually controlled, but scum and sludge pumps will be operated both automatically or manually. The clarifier should be provided with a torque limit control and an alarm system.

Scum pumps in the scum pit will be operated both automatically and manually. The automatic operation should be controlled by the water elevations in the scum pit.

Sludge pumps will be operated by timer settling. The sludge, regardless whether primary sludge only or primary plus excess sludge from the biological stage is pumped, is sent to the sludge drum screen and then to the sludge thickeners. The removal of sludge from the primary clarifiers will be controlled using timers. The pumping cycles are to be established on the basis of operational experience.

The sludge pumping from the primary clarifiers will be based on manual operation, by measuring the sludge blanket height (usually twice a day, morning and afternoon). Once the sludge blanket height has been determined, the timing cycle for sludge pumping to the sludge thickeners is manually set for each clarifier.

Degree of Instrumentation: Instrumentation will be simple and consist of a report on the running time totalizer for each of the pumps. This information will be transmitted to and recorded at the central control room. Indication of the sludge scraper mechanism operation (torque alarm) should be provided at the central control station.

3.4 BIOLOGICAL TREATMENT

Biological treatment consists of aeration tanks and final clarifers.

3.4.1 Aeration Tanks

The design parameters for this process component are established as follows:

Design inflow rate	43,000 m^3/day or 0.498 m^3/sec
Average inflow BOD_5	concentration 170 mg/l
Total BOD_5	5,117 kg/day
F/M	0.3 kg BOD_5/kg MLVSS/day
MLSS	1,667 mg/l
Hydraulic detention time	6.3 hours at the maximum daily flow
Recycle capability	50 % of maximum daily flow.
Liquid depth	5.5 m
Aeration system	Fine bubble diffusers
BOD removal efficiency	89.5 % (combined with clarifiers)

Reactor Geometry: The biological system layout will be in tank modules. The selection of aeration tank geometry is determined to make the maximum use of the available space. The contact reactor geometry is summarized as follows:

Tank width	5.5 m
Liquid depth	5.5 m
Tank length	49 m
Number of tanks	8 units
Effective tank volume	11,368 m^3

Flow Distribution: The design calls for maximum process flexibility. Hence, in the tank layout, a provision is made to make even flow distribution among all tanks, and ability to isolate individual tanks. The reactors are able to handle the maximum daily flow rate of 0.498 m^3/sec. plus maximum recycle flow of 50 percent based on the maximum daily flow.

Air Supply: The air requirements for the reactor tanks have been calculated on the basis of 0.0779 x Q (kg O_2/day). Distribution of air to the tanks expected to be even to every ditch, however, there may be some occasions when this will change. Thus, in the design of air distribution system, this will be taken into account and to have the ability to respond to such changes in air demand. The air delivery system will consist of air diffusers. The system will have the capability of maintaining a mixed liquor dissolved oxygen concentration of 1.5 mg/l or more.

Dissolved Oxygen and SS Monitors: A minimum dissolved oxygen concentration of 1.5 mg/l will be maintained in the tanks. Sensors to measure the in-situ dissolved oxygen concentration will be used.

3.4.2 Final Clarifiers

Hydraulic Loading and Area Requirements: The hydraulic loading rate for the clarifiers is 17 m^3/m^2/day at the maximum daily flow rate of 0.498 m^3/sec. Thus, the required total surface area is calculated to be 2,464 m^2.

The final clarifier geometry is summarized as follows:

Surface loading (at Q max.)	17 m^3/m^2/day
Design inflow rate	0.498 m^3/sec
Tank surface area	616 m^2 (Romanian standards)
Clarifier diameter	30 m

Effective water depth	3.5 m
Number of clarifiers	4 units

Sludge Production: The secondary sludge production rate of is as follows:

Total SS	5.11 ton/day
Sludge concentration	0.5 %
Sludge volume	1,023 m^3/day

The excess sludge is sent either directly from the clarifiers or through the primary clarifiers to the sludge thickeners. In the latter case, the excess sludge is pumped from the clarifiers to the flow distribution chamber of each clarifier. The excess sludge settles with the primary sludge and then pumped to anaerobic sludge digesters.

Return Sludge Pumps: The pump capacity is determined based on the maximum 100 % sludge return ratio of the reactor tank inflows. The pump design parameters are as follows:

No.1 pumps

Pump type	Centrifugal screw pump
Pump diameter	150 mm
Capacity	2.3 m^3/min.
TDH	10 m
Number of pumps	4 sets
Motor output	7.5 kW

No.2 Pumps

Pump type	Centrifugal screw pump
Pump diameter	150 mm
Capacity	3.0 m^3/min.
TDH	10 m
Number of pumps	2 sets
Motor output	11 kW

No.3 Pumps

Pump type	Centrifugal screw pump
Pump diameter	250 mm
Capacity	8.0 m^3/min.
TDH	10 m
Number of pumps	2 sets (for additional 50% sludge return)
Motor output	30 kW

Excess Sludge Pumps: The excess sludge of 1,023 m^3/day or 0.7 m^3/min. will be sent either directly or through the primary clarifiers to the sludge thickeners. The criteria for the pump equipment are as follows:

Pump type	Centrifugal screw pump
Pump diameter	100 mm
Capacity	1.0 m^3/min.
TDH	10 m
Number of pumps	2 sets (including 1-standby)
Motor output	3.7 kW

Controls: Clarifier operation will be controlled manuallyn, whereas the sludge pumps will be controlled both automatically or manually. The clarifiers should be provided with torque limit control and an alarm system.

The sludge pumping from the secondary clarifiers is made on manually measuring the sludge blanket height. Once the sludge blanket height has been determined, the timing cycle for sludge pumping to primary clarifiers or thickener facilities is manually set for each clarifier. The pumps turn on/off sequentially based on the on-off time increments for each of the clarifiers. There would be readout showing that pumps are on and the individual pump running times.

Degree of Instrumentation: Instrumentation will consist of a report on the running time totalizer for each of the pumps. This information will be transmitted to and recorded at the control room. Indication of the sludge scraper mechanism operation (torque alarm) will be provided at the control station.

3.4.3 Chlorine Contact Tanks

Chlorination System: For disinfection of the wastewater before its discharge to watercourses, the disinfection system capacity is sufficient to produce an effluent that will meet the coliform bacteria limits specified by the standards for that installation at all time(e.g. total coliform bacteria and fecal coliform bacteria numbers are 1 million and 10,000 MPN/100 ml, respectively). This condition must be attainable when maximum flow rates occur and during emergency conditions.

The solution chlorine disinfection system consists of contact tank, chlorination equipment, housing and storage, and ancillary services. Means of removal of solids from the tank bottom are to be provided. Skimming devices will be provided in all contact tanks.

Design Parameters: The design parameters for the chlorine contact tank are as follows:

Hydraulic maximum flow rate	43,000 m^3/day
Contact time	15 minutes at the design rate of flow
Capacity of chlorine feed system	3 mg/l at the maximum flow rate
Hypochlorite storage capacity	12.4 m^3 (8 days)

Duplicate disinfection systems will be provided. Where only two units are installed, each will be capable of feeding the expected maximum dosage rate.

Contact Tank Geometry: The primary purpose of the contact tank is to provide the detention time necessary for the chlorine compounds to reduce the bacteria to acceptable level. The design aim is to minimize the short-circuiting and dead spaces through basin configuration and flow pattern control. The chlorine contact tank will be of reinforced concrete longitudinal baffled basin, which will have a large effective length-to-width ratio. The contact tank geometry is summarized below:

Number of tank units	1 basin
Channel width	4 m
Channel depth	4 m
Channel length	38 m
Tank effective volume	448 m^3
Effective water depth	3 m

Equipment: The installed capacity of a chlorine feed system will be sufficient to provide a dosage of 3 milligrams per liter at the maximum design rate of flow. The feed equipment will consist of the following

Solution storage tank

Type	FRP cylinder type
Internal diameter	1,800 mm
Height	2,900 mm
Tank capacity	6 m^3
Number of tanks	2 units

Feed pumps

Type	Diaphragm pump
Discharge capacity	0.5 L/min.
Motor output	0.4 kW
Number of pumps	3 units (including one standby)

Housing and Storage: Local, state and federal safety requirements, including fire code, will be carefully followed in storing and handling of chlorine containers, cylinders or tank cars.

Ventilation: Forced, mechanical ventilation is to be installed which will provide one complete air change per minute when the room is occupied. Adequate provisions will be made to insure that one complete air change per minute is provided when the room is occupied.

3.4.4 Effluent Pumping Station

The pumping station is in principle of the same type of the influent pumping station, with dry-well and wet-well.

Provision shall be made to facilitate easy removing of pumps, motors and other auxiliary equipment. Suitable safe means of access should be designed to the dry well of the pumping station, including stairways, handrails and gratings where necessary.

Adequate ventilation should be considered for the wet- and dry wells. For the pump room floor below the ground surface, mechanical ventilation is provided, so arranged as to independently ventilate the dry well. The wet-wells will be open and no mechanical ventilator will be provided.

Pump Equipment and Operation Control: Totally 4 units of pump are planned, including one-standby. The pumps are designed to have the same capacity and size, with the sufficient capacity for handling the flow in excess of the estimated maximum inflow.

The pump sizes, numbers and capacities of the wastewater pumps are as follows:

Type of pumps	Vertical centrifugal mixed flow pump
Pump diameter	400 mm
Pump discharge capacity	15 m^3/min
TDH	5.5 m
Number of pump	4 units (one standby)
Motor output	21 kW

Wet-Well (Pump Well): The wet-well should be divided into two separate compartments for the convenience of cleaning and inspection of the well.

In order to prevent unnecessary vortices forming in the well, particular considerations are to be given to the bottom slopes and arrangement of suction pipes, thereby providing optimum hydraulic condition for the pump operation.

The wet-well size is to be determined considering the required space between pumps and proper pump suction conditions. The shape of the well and the detention is determined so as to minimize deposition of solids and prevent the wastewater to become septic.

Dry-Well (Pump Room): In the dry-well, sufficient room is to be maintained between pumps to move the pump off its base with ample clearance left over between suction and discharge piping, and room for on site repairs, inspection, or removal from the room to the surface for repairs.

The size of dry well is to be determined based on the number and type of pumps selected and piping arrangement. A sufficient space between the pumps should be taken each at the both end of the room. The width should provide a sufficient space for the required length for pipes, valves and clearance for maintenance and repairs.

All safety and other requirements are also to be implemented in accordance with local and national safety codes and regulatory agencies. Provisions should also be made for drainage from pump water seal connections.

Piping and Valves: Suction, discharge and header piping in the station are sized to handle the flows adequately. Pipes are sized so that the velocity in the suction line should not exceed 1.5 m/sec, and the discharge piping 2.4 m/sec.

Valves are to be provided on the suction and discharge side of each pump to allow proper maintenance of the unit. To the discharge pipeline, electric motor-operated butterfly valves and the check valves shall be installed to ensure the operation of each pump.

Hoisting Equipment: An electric overhead bridge traveling crane should be provided in the motor room for handling of equipment and materials which cannot be lifted readily or removed from the station by manual labor. The crane should be provided in the motor room for handling of equipment and materials, which cannot be lifted readily or removed from the station by manual labor.

3.5 SLUDGE MANAGEMENT

3.5.1 Gravity Sludge Thickeners

The design of gravity sludge thickeners should consider the type and concentration of sludge, the sludge stabilization processes, the method of ultimate sludge disposal, chemical needs, and the cost of operation. The pumping rate and piping of the concentrated sludge should be selected such that anaerobic conditions are prevented.

Design Basis: Equipment and piping are to be designed to deliver sufficient dilution water to gravity thickeners. Hydraulic loading to produce overflow rates of 16~33 $m^3/m^2/day$ will be maintained to prevent septicity.

The loading rates and resulting solids concentration for gravity thickening are as follows:

Average sludge production volume	1,169 m^3/day
Sludge withdrawal rate	184 m^3/day
Input sludge solids	8.04 t/day

SS loads	60 kg/m^2/day

Tank Geometry: Thickener tanks geometry is summarized as follows:

Tank shape	Circular
Number of tanks	2 units
Internal diameter	9.5 m
Effective water depth	4 m
Effective tank surface area	142 m^2
Thickening mechanism	Rotating type scraper supported by center column with Tickets

Equipment Features: Heavy-duty scrapers capable of withstanding extra heavy torque loads should be provided. The thickener mechanism may be provided with pickets to help facilitate the release of water from the sludge. The drive mechanisms will be attached with a skimmer. The collected scum will be discharged into a central scum pit located at the thickeners. Ability to add chlorine solution should be provided to prevent septicity.

Sludge Pumps: The pump capacity is so determined that the pumps can send the thickened sludge within 8 hours. Specifications of these equipment are as follows:

Type	Sludge pump with suction screw
Number of pumps	2 sets (one standby)
Diameter	100 mm
Discharge capacity	1.2 m^3/min.
TDH	20 m
Motor output	15 kW

Drum Screen: Prior to pumping the primary or secondary excess sludge to the sludge thickeners, the sludge will be screened by a revolving drum screen for the removal of coarse materials. The sludge pumps send the sludge to the drum screen and then screened sludge flows into the sludge thickeners. All removed coarse materials will be dumped from the screen and collected in a bin that will be emptied manually into trucks and hauled to sanitary landfill for final disposal. The specifications of the drum screen are as follows:

Type	Rotary drum screen
Number of screen	1 set
Screen openings	4mm
Screening capacity	2 m^3/min.
Motor output	0.4 kW

Controls: Tank operation will be manually controlled, but sludge pump operation will be controlled both automatically or manually. Pickets will be provided with torque limit control and an alarm system.

The sludge pumps will be operated using on/off pump controls and timers. The sludge blanket height will be determined manually. Flow rate of dilution water when used will be measured and recorded.

3.5.2 Anaerobic Digestion Tanks

Digestion Process: Active digestion, concentration and storage will undergo in a single-stage anaerobic digestion tank. There will be a cluster consisting of 2 independent digestion tanks. Mechanical mixing system, heating and gas collection systems should be installed in each tank.

The concentrated sludge will be pumped to the digestion tanks after passing through a drum screen. The digested sludge is withdrawn by gravity to the storage tanks in the sludge dewatering building. The produced gas will be led to gasholders after passing through gas scrubbers for the use of boilers.

Design Basis: The total digestion tank capacity is determined by the calculations based upon the following factors:

Input sludge solids	6.43 t/day
Sludge input	184 m^3/day
Sludge output	139 m^3/day
Temperature to be maintained in the digesters	35 ℃
Solid detention time	20 days
The degree and extent of mixing in the digesters	Moderate mixing
Required total tank capacity(2 tanks)	4,030 m^3

Tank Geometry: The tank shape will be of high vertical cylinder with conical floors. The digester tank capacity is calculated based on the estimated sludge production in 2010. The total number of anaerobic digestion tanks required is 2 tanks in two clusters, with the same capacity and configuration. Tank dimensions are as follows:

Tank shape	Single stage, high vertical cylinder with conical floors
Tank capacity	2,015 m^3
Tank diameter	12.5 m
Tank effective water depth	21 m

Sludge Inlets and Outlets: Recirculation, withdrawal and return points should be provided, to enhance flexible operation and effective mixing. The returns should be discharged above the liquid level and be located near the center of the tank. Raw sludge feed to the digesters should be made either through heat exchangers or directly to the tank.

Sludge withdrawal to dewatering system and disposal shall be from the bottom of the tank. This pipe should be interconnected with the recirculation piping, to increase versatility in mixing the tank contents. Additional alternative withdrawal lines should be provided.

Sampling: Sampling hatches shall be provided in all tank covers with water seal tubes extending to beneath the liquid surface.

Mixing Systems: Sludge mixing systems should be mechanical recirculation type. The mixing system shall be designed such that routine maintenance can be performed without taking the digester out of service.

3.5.3 Sludge Gas System

Gas Collection, Piping and Appurtenances: All portions of the gas system, including the space above the tank liquor, storage facilities and piping, are so designed that under normal operating conditions, including sludge withdrawal, the gas will be maintained under positive pressure. All enclosed areas where any gas leakage might occur should be adequately ventilated. Gas meters, with by-pass, should be provided to meter total and waste gas production.

All safety equipment shall be provided where gas is produced. Pressure and vacuum relief valves, flame traps, gas detectors, and automatic safety shut off valves, shall be provided.

Gas Utilization Equipment: Gas-fired boilers for heating digesters will be located in a separate room not directly connected to the digester gallery. Gas lines to these units shall be provided with flame traps. Gas piping shall be of adequate diameter for gas flow rate and will slope to condensate traps at low points.

Electrical Fixtures: Electrical fixtures and controls in enclosed places where hazardous gases may accumulate shall comply with the local or national codes. Digester galleries should be isolated from normal operating areas to avoid an extension of the hazardous location.

Waste Gas: Waste gas burners should be readily accessible and be located at least 8 meters away from any plant structure if placed at ground level, or they may be located on the roof of the control building at a height of not less than 0.9 m from the top of the roof.

All waste gas burners should be equipped with automatic ignition, such as a pilot light or a device using a photoelectric cell sensor. Necessary approvals from the local authorities concerned shall be obtained for burning any waste gas and any other emissions from the treatment plant.

Any underground enclosures connecting with digesters or containing sludge or gas piping or equipment should be forced ventilated. The piping gallery for digesters should not be connected to other passages.

Digester Heating: Digesters should be constructed above ground water level and suitably insulated to minimize heat loss.

Sludge will be heated by circulating the sludge through external heaters. Piping may be designed to provide for the preheating of feed sludge before introduction to the digesters. Provisions should be made in the layout of the piping and valving to facilitate cleaning of these lines.

The boiler should be provided with suitable automatic controls to maintain the boiler temperature at a fixed rate, to minimize corrosion, and to shut off the main gas supply in the event of pilot burner or electrical failure, low boiler water level, or excessive temperatures. Thermometers is to be provided to show temperatures of the sludge, hot water feed, hot water return, and boiler water.

Access Manholes: At least two 90-cm diameter access manholes shall be provided in the top of the tank in addition to the gas dome. There should be stairways to reach the access manholes.

Safety: Local, state and federal safety requirements must be reviewed and complied with. Those requirements take precedence over the requirements stated herein, if more stringent, and should be incorporated in the design. Non-sparking tools, safety lights, rubber-soled shoes, safety harness, gas detectors for inflammable and toxic gases, and at least two self-contained breathing units will be provided for emergency use.

3.5.4 Gas Holder

The digestion gas is led to gasholders via gas scrubbers, whereby the gas will be de-sulfarized and used for the boilers. Waste gas burners should be provided for the case of gas boilers malfunction. The gas production is estimated assuming that 70 % of the input solid components are biodegradable organic materials, 1 kg of which produces 0.425 m^3 of sludge gas. Thus, the total daily gas production is estimated to be 1,913 m^3. The gas holder will have the minimum gas storage capacity of 8-hour gas production.

The geometry of the gas holder is as follows:

Type	dry seal gasholder (membrane seal type)
Number of unit	1 tank
Diameter	11.6 m
Height	9.2 m
Capacity	638 m^3

3.5.5 Belt Filter Press Sludge Dewatering

The daily digested sludge of 801 m^3 is drawn by gravity into the storage tanks located in the dewatering building. The digested sludge production rate and the required dewatering equipment are as summarized in the following:

Total digested sludge production	139 m^3/day
Total sludge solids	4.18 t/day
Dewatering equipment	belt filter press
Yields per unit length	130 kg/m/hr.
Filter width	2 m
Daily operation time	6 hours
Working days per week	5 days
Solids load per hour	975 kg
Required filter press equipment	4 units

An air cylinder operated eccentric valve will be installed on the line between the digester and the storage tank. The sludge from the storage tank is pumped to the coagulation tank of the dewatering equipment (belt filter press) by sludge feed pumps.

The coagulation tank is to mix the polymer with the sludge. On the other hand, polymer solution are mixed and then pumped into the coagulation tank of the belt filter press. For the above, dry polymer is stored in hoppers, and led into dosage tanks by metering feeders and finally dissolved and stored in the dosage tanks. The polymer system shall have units of motorized hoist for unloading chemicals from the truck.

The dewatered sludge (sludge cake) is conveyed to the cake yard by the trough belt conveyors. The filtrate, together with the belt filter cleansing wastewater is returned to the process wastewater storage tanks by gravity and then pumped to the inlet of the screen chamber by process wastes return pumps. Two units of electrically operated overhead crane are provided in the building for dismantling and repairing the dewatering equipment.

3.5.6 Process Wastewater Return Pump Facilities

The process wastewater return pump system is to return the process wastes (i.e. building wastewater, digester supernatant, belt press filtrate and scum filtrate) to the screen inlet chamber for further treatment.

3.6 MAJOR MECHANICAL EQUIPMENT

As discussed previously, a number of mechanical equipment are required for the Tulcea WWTP. The number, types, functions, and specifications of the major mechanical equipment to be applied are outlined and listed in *Table AII.3.1*. It should be noted, however, that the equipment described here are those estimated for the preliminary engineering purposes and are subject to minor changes at the detailed design stage.