7.3 Transmission Facility

The following matters are considered in the preparation of preliminary engineering design of transmission line in each alternative. Drawings for each alternative are presented in FIGUREs 7-7 to 7-15.

(1) Design Sewage Flow

In designing the conduit, the planned intake flow of each alternative is used for determination of conduit section.

The fluctuation of intake amount (refer to subsection 7.2.3) was taken into consideration for the decision of diameter and slope of transmission pipe to convey the planned sewage amount without hindrance.

(2) Flow calculation

The Manning's formula is applied for the gravity flowing section with free water surface:

Manning's Formula: $Q = A.\ V$

$$V = (1/N)\ R^{(2/3)} S^{(1/2)}$$

where;

- Q: Quantity of flow (m^3/sec)
- A: Flow area (m^2)
- V: Flow velocity (m/sec)
- N: Coefficient of roughness
  for Concrete Pipes - 0.013
- R: Hydraulic radius (m) (=A/P)
- P: Wetted perimeter
- S: slope

On the other hand, the Hazen - Wiliams' Formula is applied to the pressurized flowing section such as inverted siphon sections and pressurized flow sections :

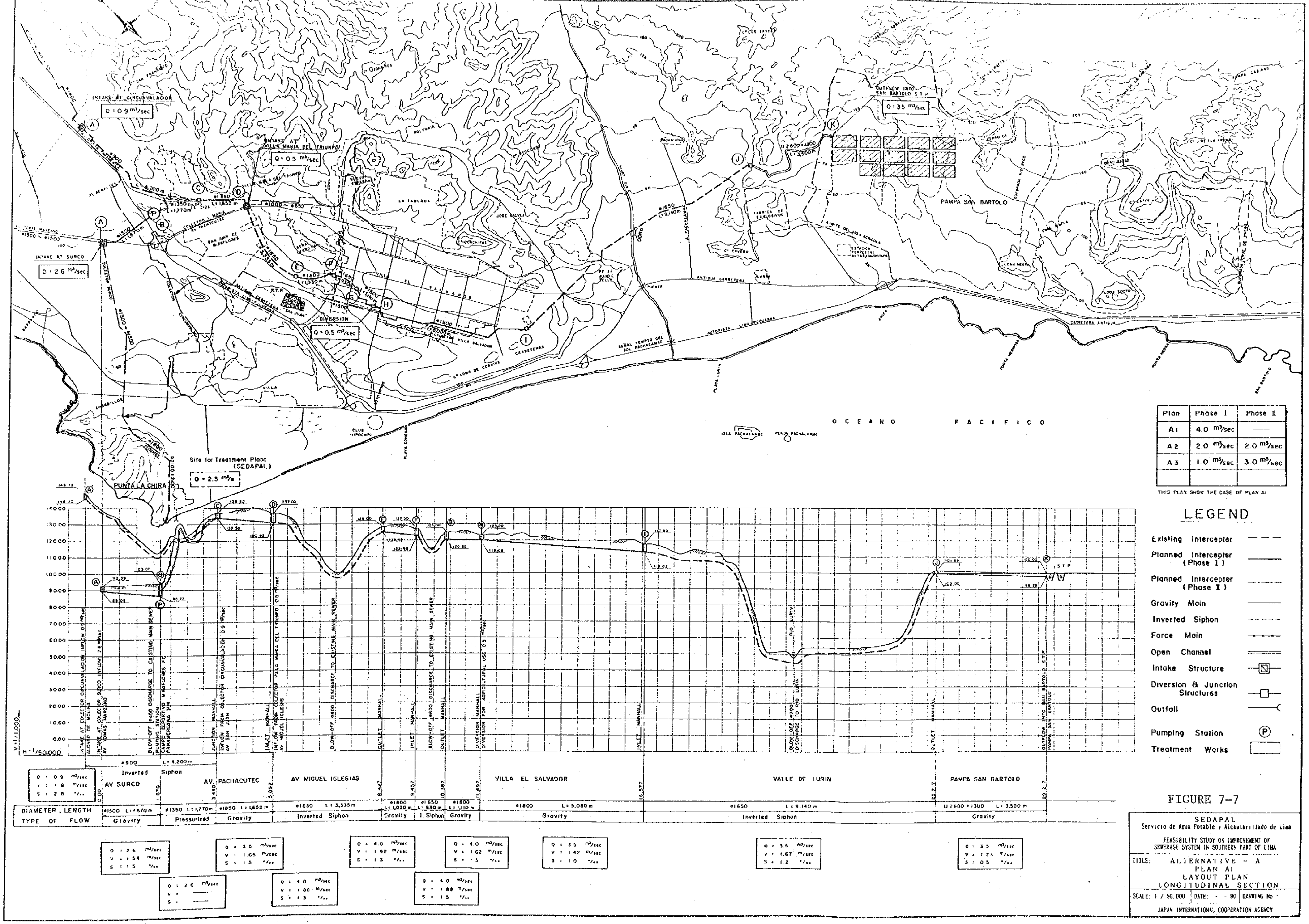
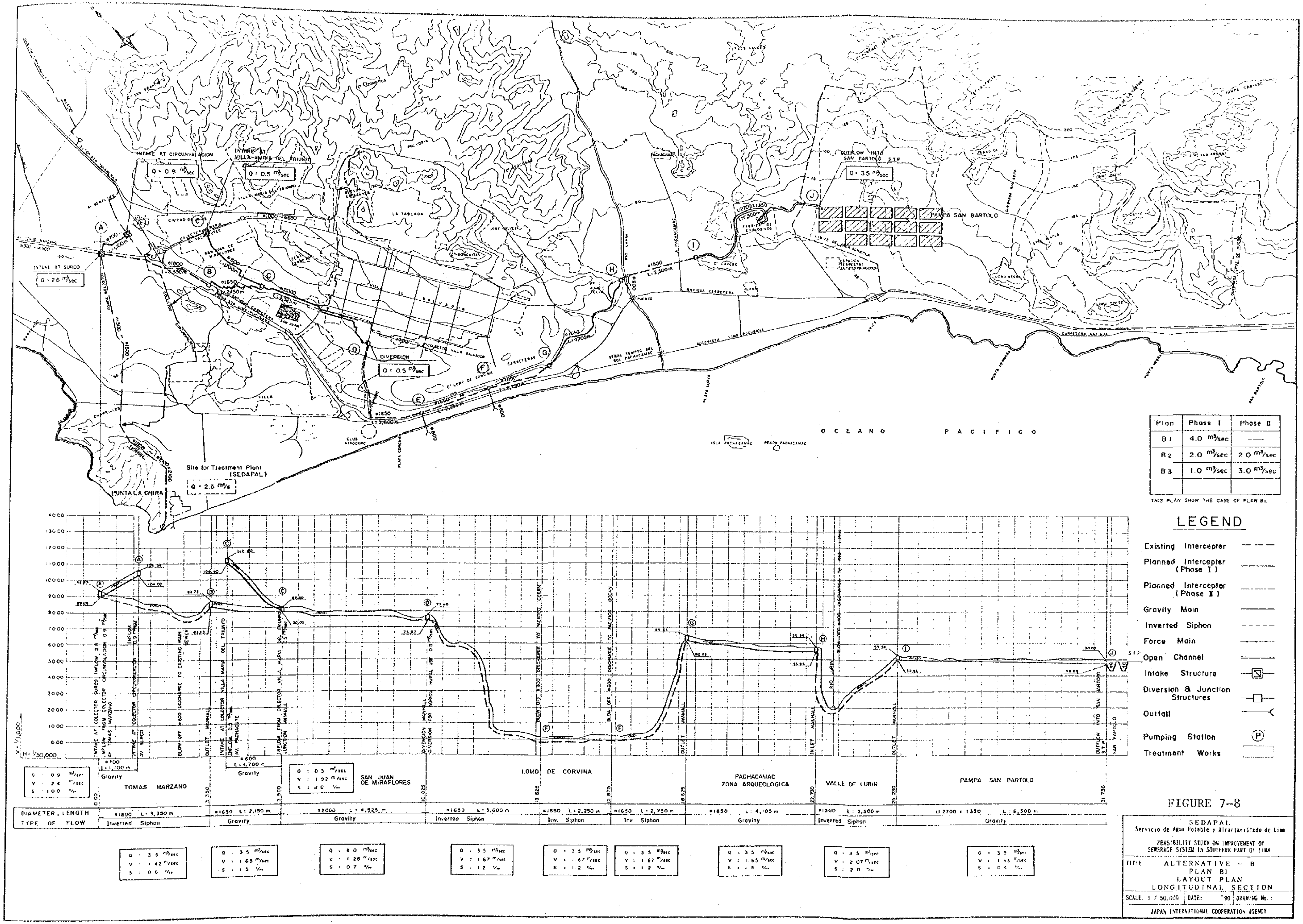

INTAKE AT CIRCUNVALACION
Q = 0.9 m³/sec
INTAKE AT VILLA MARIA DEL TRIUNFO
Q = 0.5 m³/sec
INTAKE AT SURCO
Q = 2.6 m³/sec
DIVERSION
Q = 0.5 m³/sec
OUTFLOW INTO SAN BARTOLO S.T.P
Q = 3.5 m³/sec
PAMPA SAN BARTOLO
VILLA EL SALVADOR
LA TABLADA
OCEANO PACIFICO
Site for Treatment Plant (SEDAPAL)
Q = 2.5 m³/s
PUNTA LA CHIRA
Plan
Phase I
Phase II
A1
4.0 m³/sec
A2
2.0 m³/sec
2.0 m³/sec
A3
1.0 m³/sec
3.0 m³/sec
THIS PLAN SHOW THE CASE OF PLAN A1
LEGEND
Existing Intercepter
Planned Intercepter (Phase I)
Planned Intercepter (Phase II)
Gravity Main
Inverted Siphon
Force Main
Open Channel
Intake Structure
Diversion & Junction Structures
Outfall
Pumping Station
Treatment Works
V 1/1,000
H 1/50,000
AV. SURCO
AV. PACHACUTEC
AV. MIGUEL IGLESIAS
VILLA EL SALVADOR
VALLE DE LURIN
PAMPA SAN BARTOLO
DIAMETER, LENGTH
TYPE OF FLOW
Gravity
Pressurized
Gravity
Inverted Siphon
Gravity
I. Siphon
Gravity
Gravity
Inverted Siphon
Gravity
SEDAPAL
Servicio de Agua Potable y Alcantarillado de Lima
FEASIBILITY STUDY ON IMPROVEMENT OF SEWERAGE SYSTEM IN SOUTHERN PART OF LIMA
TITLE: ALTERNATIVE - A
PLAN A1
LAYOUT PLAN
LONGITUDINAL SECTION
SCALE: 1 / 50,000
DATE: - '90
DRAWING No.:
JAPAN INTERNATIONAL COOPERATION AGENCY

FIGURE 7-7

A1



Plan	Phase I	Phase II
B1	4.0 m³/sec	——
B2	2.0 m³/sec	2.0 m³/sec
B3	1.0 m³/sec	3.0 m³/sec

THIS PLAN SHOW THE CASE OF PLAN B1.

LEGEND

- Existing Interceptor
- Planned Interceptor (Phase I)
- Planned Interceptor (Phase II)
- Gravity Main
- Inverted Siphon
- Force Main
- Open Channel
- Intake Structure
- Diversion & Junction Structures
- Outfall
- Pumping Station
- Treatment Works

FIGURE 7-8

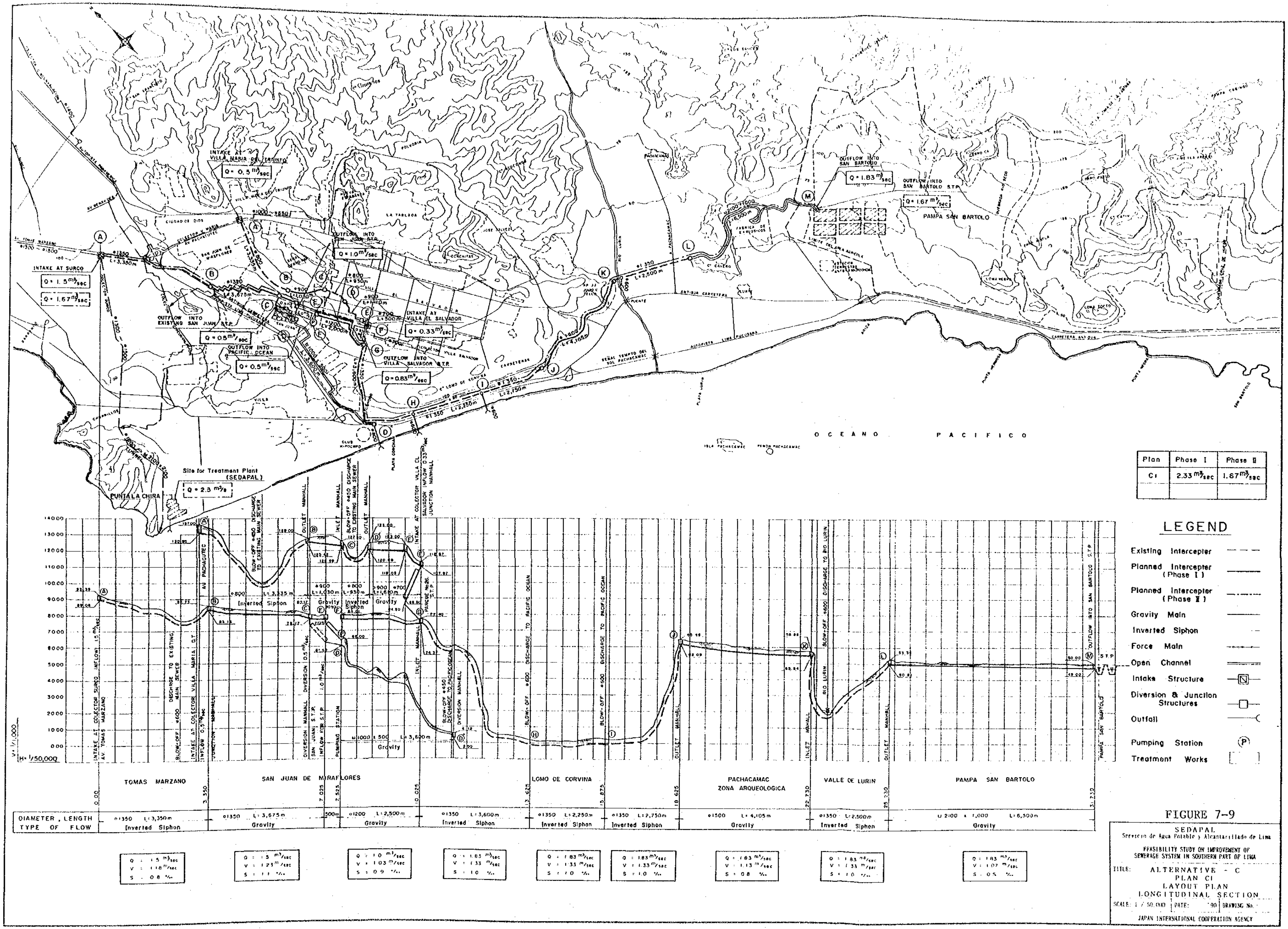
SEDAPAL
Servicio de Agua Potable y Alcantarillado de Lima

FEASIBILITY STUDY ON IMPROVEMENT OF SEWERAGE SYSTEM IN SOUTHERN PART OF LIMA

TITLE: ALTERNATIVE - B
PLAN B1
LAYOUT PLAN
LONGITUDINAL SECTION

SCALE: 1 / 50,000 DATE: - '90 DRAWING No.:

JAPAN INTERNATIONAL COOPERATION AGENCY


OCEANO PACIFICO
INTAKE AT SURCO
INTAKE AT VILLA MARIA DEL TRIUNFO
OUTFLOW INTO EXISTING SAN JUAN S.T.P.
OUTFLOW INTO PACIFIC OCEAN
INTAKE AT VILLA EL SALVADOR
OUTFLOW INTO VILLA SALVADOR S.T.P.
OUTFLOW INTO SAN BARTOLO
OUTFLOW INTO SAN BARTOLO S.T.P.
PAMPA SAN BARTOLO
Site for Treatment Plant (SEDAPAL)
PUNTA LA CHIRA
Plan | Phase I | Phase II
C1 | 2.33 m3/sec | 1.67 m3/sec
LEGEND
Existing Intercepter
Planned Intercepter (Phase I)
Planned Intercepter (Phase II)
Gravity Main
Inverted Siphon
Force Main
Open Channel
Intake Structure
Diversion & Junction Structures
Outfall
Pumping Station
Treatment Works
TOMAS MARZANO
SAN JUAN DE MIRAFLORES
LOMO DE CORVINA
PACHACAMAC ZONA ARQUEOLOGICA
VALLE DE LURIN
PAMPA SAN BARTOLO
DIAMETER, LENGTH
TYPE OF FLOW
Inverted Siphon
Gravity
V 1/1,000
H 1/50,000
FIGURE 7-9
SEDAPAL
Servicio de Agua Potable y Alcantarillado de Lima
FEASIBILITY STUDY ON IMPROVEMENT OF SEWERAGE SYSTEM IN SOUTHERN PART OF LIMA
TITLE: ALTERNATIVE - C
PLAN C1
LAYOUT PLAN
LONGITUDINAL SECTION
SCALE: 1 / 50,000
DATE: '90
DRAWING No.
JAPAN INTERNATIONAL COOPERATION AGENCY

C1

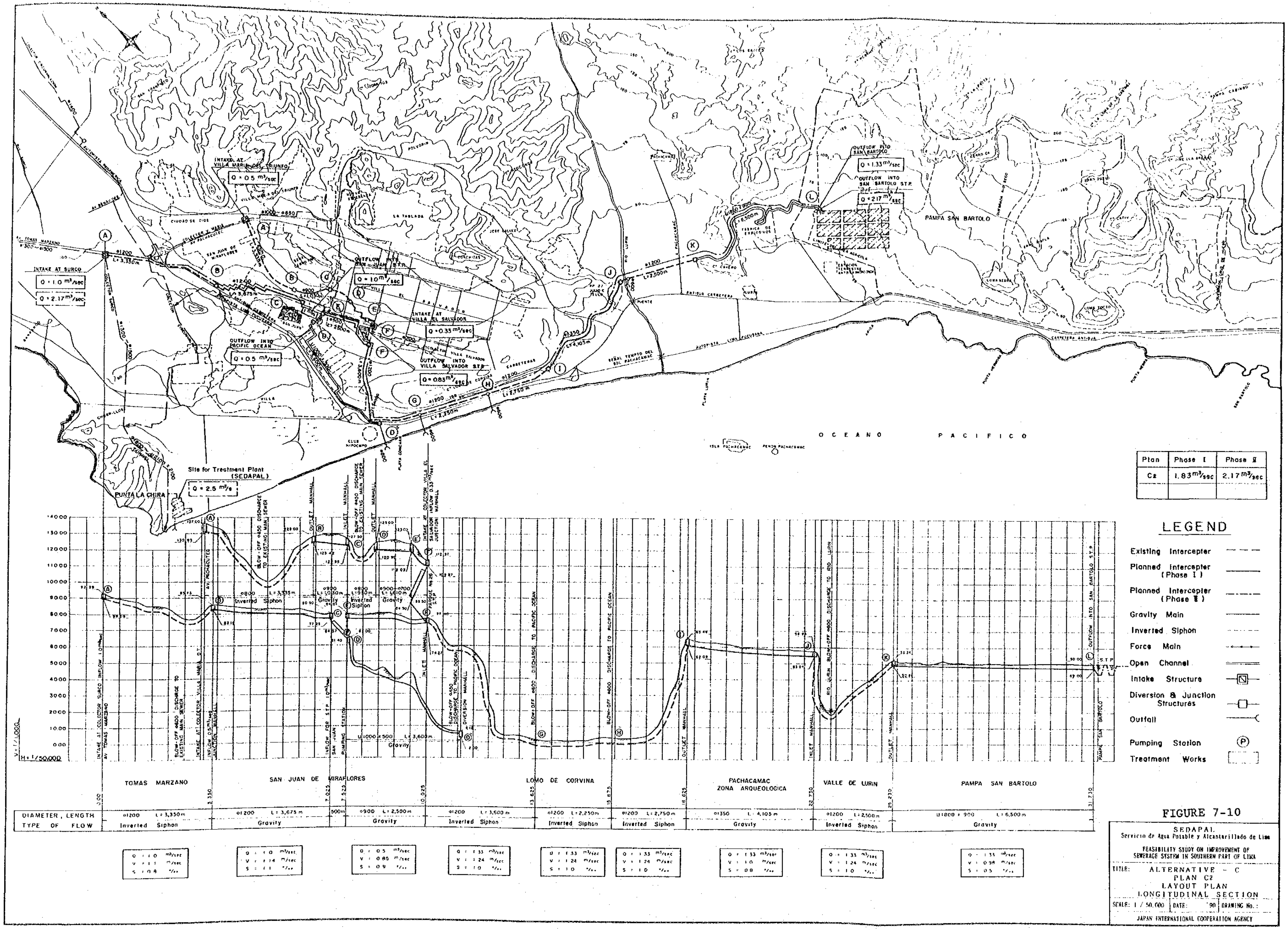


FIGURE 7-10

C2

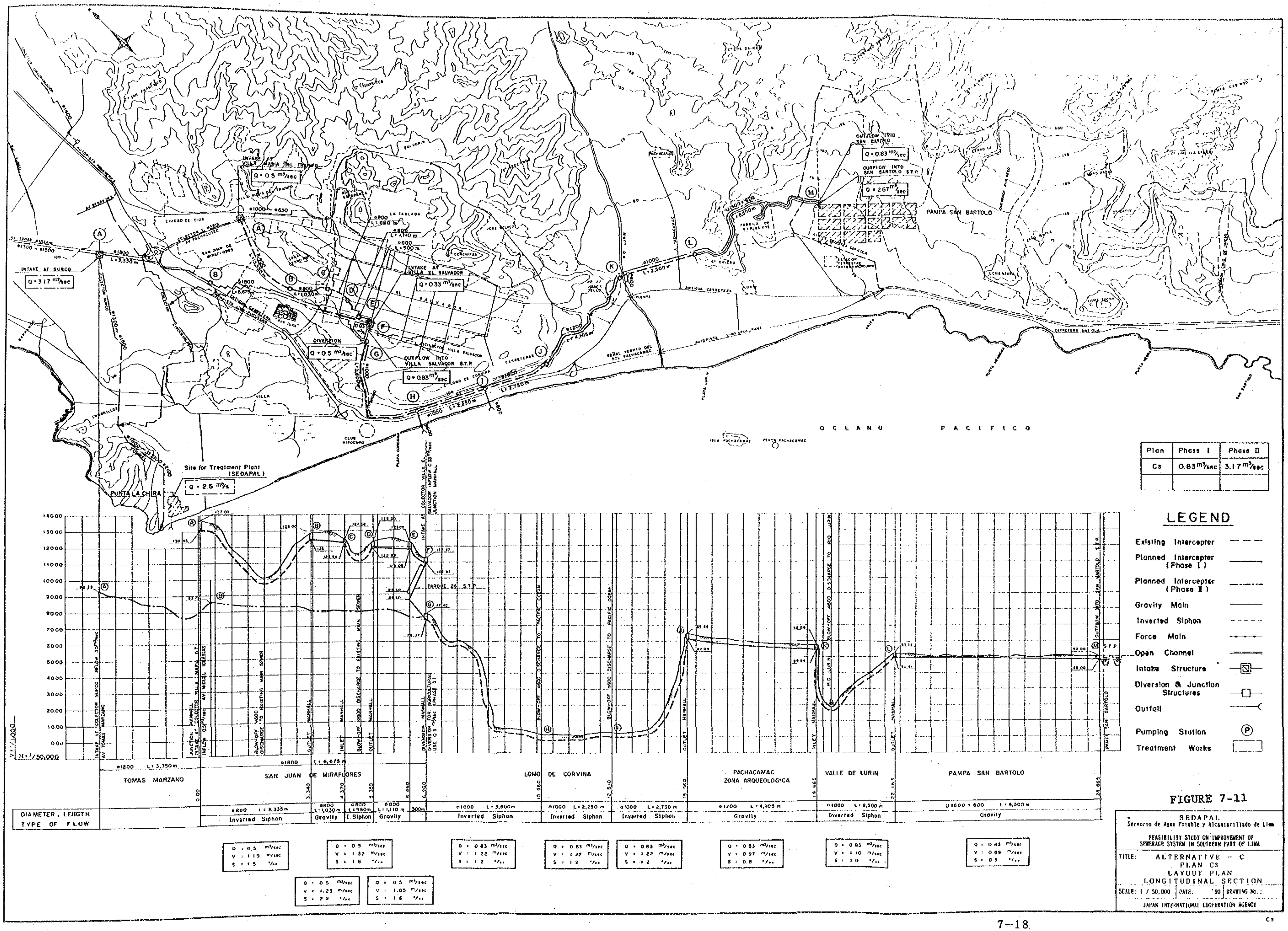

OCEANO PACIFICO
PAMPA SAN BARTOLO
INTAKE AT SURCO
Q = 3.17 m³/sec
INTAKE AT VILLA MARIA DEL TRIUNFO
Q = 0.5 m³/sec
INTAKE AT VILLA EL SALVADOR
Q = 0.33 m³/sec
DIVERSION
Q = 0.5 m³/sec
OUTFLOW INTO VILLA SALVADOR S.T.P.
Q = 0.83 m³/sec
OUTFLOW INTO SAN BARTOLO
Q = 0.83 m³/sec
OUTFLOW INTO SAN BARTOLO S.T.P.
Q = 2.67 m³/sec
Site for Treatment Plant (SEDAPAL)
Q = 2.5 m³/s
PUNTA LA CHIRA
Plan | Phase I | Phase II
C3 | 0.83 m³/sec | 3.17 m³/sec
LEGEND
Existing Interceptor
Planned Interceptor (Phase I)
Planned Interceptor (Phase II)
Gravity Main
Inverted Siphon
Force Main
Open Channel
Intake Structure
Diversion & Junction Structures
Outfall
Pumping Station
Treatment Works
V = 1/1,000
H = 1/50,000
TOMAS MARZANO
SAN JUAN DE MIRAFLORES
LOMO DE CORVINA
PACHACAMAC ZONA ARQUEOLOGICA
VALLE DE LURIN
PAMPA SAN BARTOLO
DIAMETER, LENGTH
TYPE OF FLOW
Inverted Siphon
Gravity
I. Siphon
Gravity
Inverted Siphon
Inverted Siphon
Inverted Siphon
Gravity
Inverted Siphon
Gravity
SEDAPAL
Servicio de Agua Potable y Alcantarillado de Lima
FEASIBILITY STUDY ON IMPROVEMENT OF SEWERAGE SYSTEM IN SOUTHERN PART OF LIMA
TITLE: ALTERNATIVE – C
PLAN C3
LAYOUT PLAN
LONGITUDINAL SECTION
SCALE: 1 / 50,000
DATE: '90
DRAWING No.:
JAPAN INTERNATIONAL COOPERATION AGENCY
C3

FIGURE 7-11

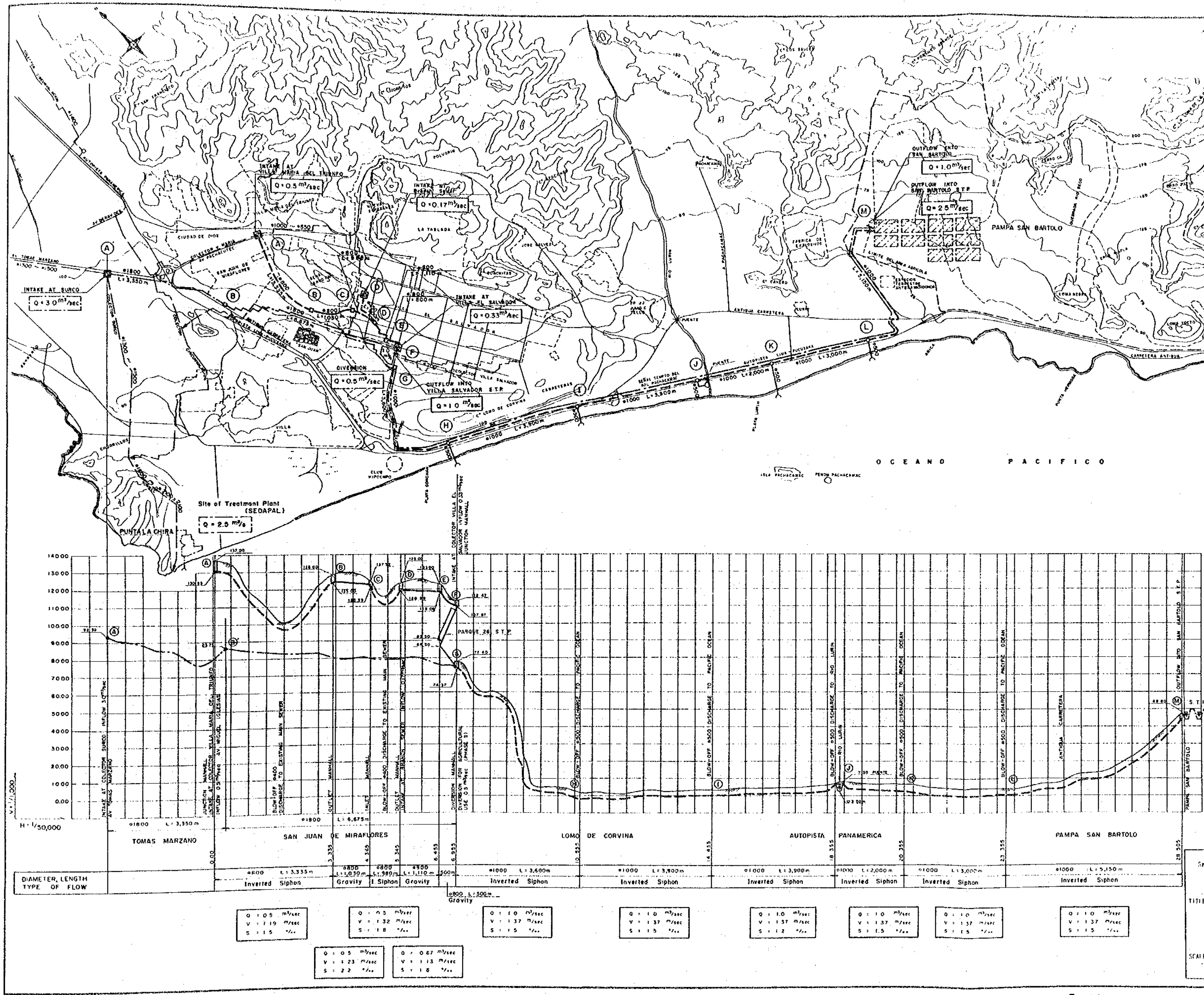


FIGURE 7-12

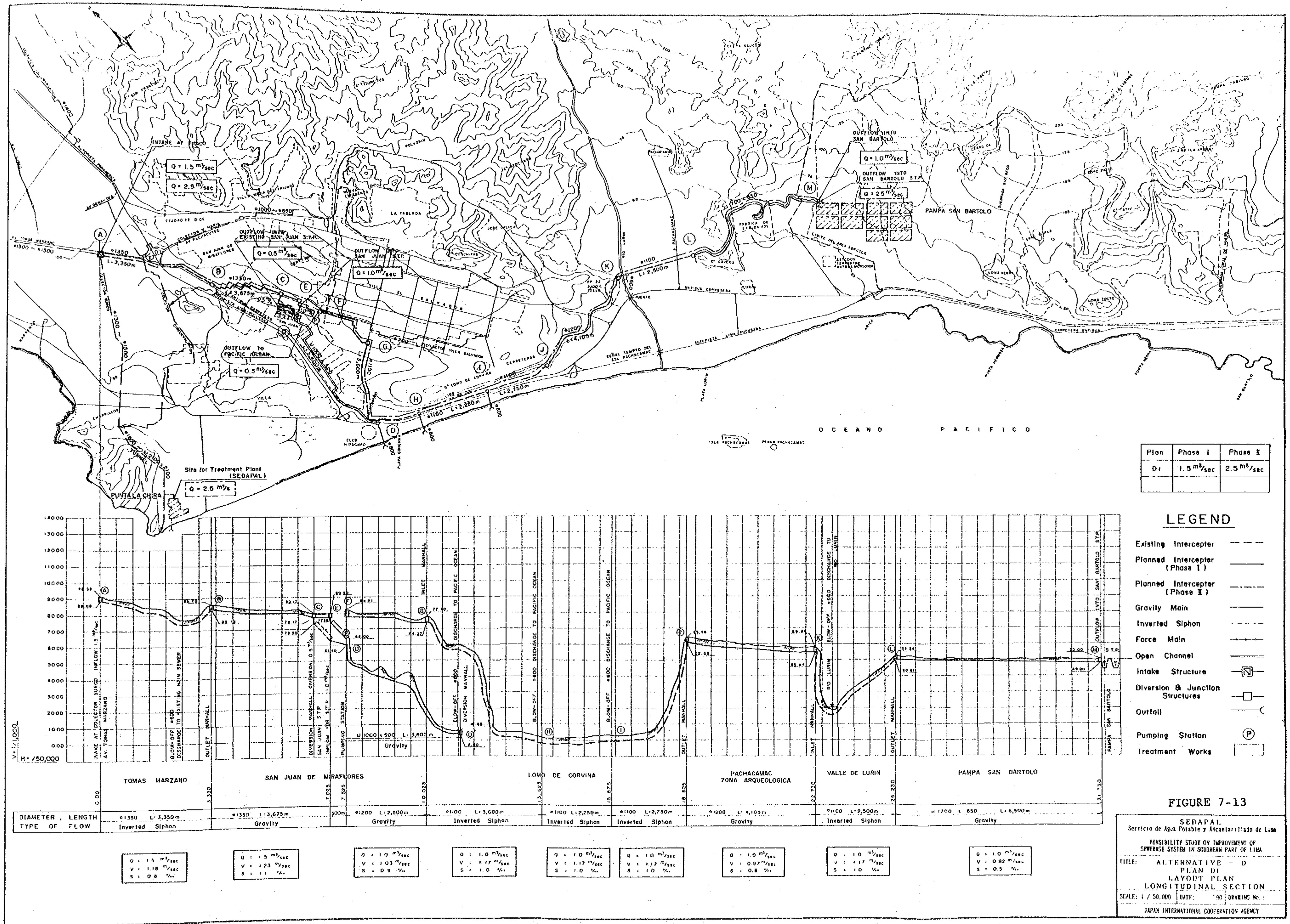
SEDAPAL
Servicio de Agua Potable y Alcantarillado de Lima

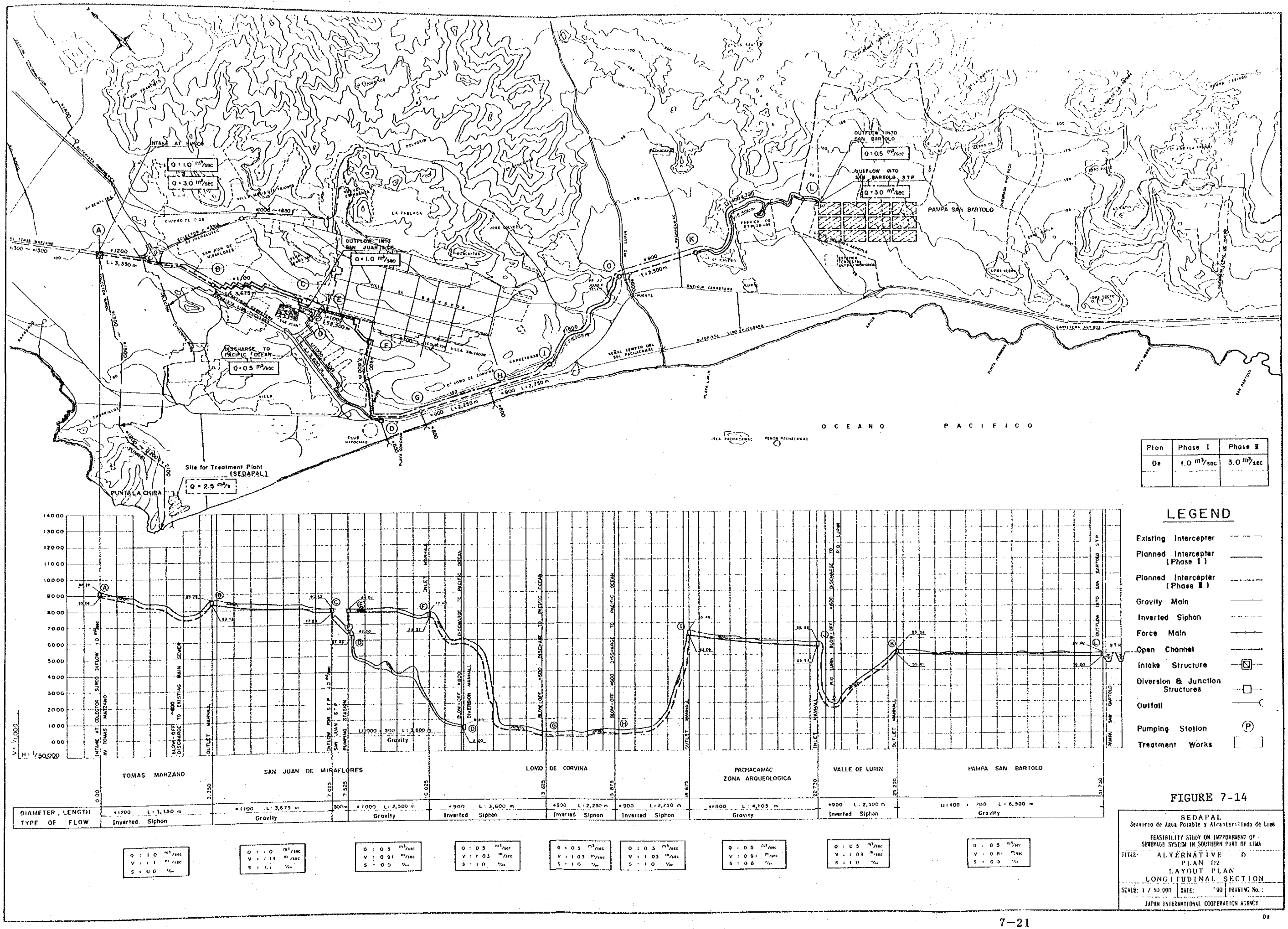
FEASIBILITY STUDY ON IMPROVEMENT OF SEWERAGE SYSTEM IN SOUTHERN PART OF LIMA

TITLE ALTERNATIVE – C
PLAN C3'
LAYOUT PLAN
LONGITUDINAL SECTION

SCALE 1/50,000 | DATE '90 | DRAWING No

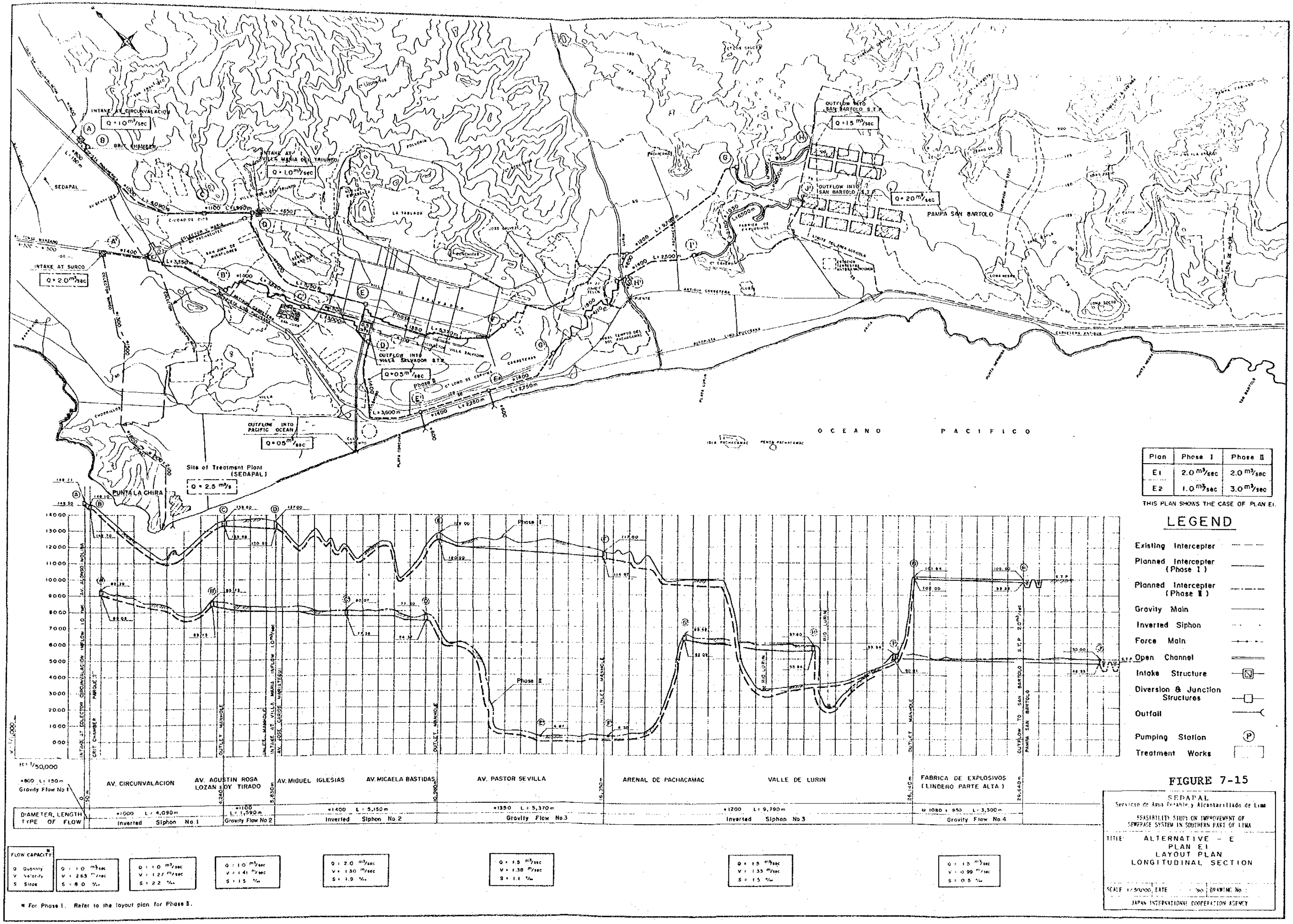
JAPAN INTERNATIONAL COOPERATION AGENCY


INTAKE AT SURCO
Q = 1.5 m³/sec
Q = 2.5 m³/sec
OUTFLOW INTO EXISTING SAN JUAN S.T.P.
Q = 0.5 m³/sec
OUTFLOW INTO SAN JUAN S.T.P.
Q = 1.0 m³/sec
OUTFLOW TO PACIFIC OCEAN
Q = 0.5 m³/sec
Site for Treatment Plant (SEDAPAL)
Q = 2.5 m³/s
PUNTA LA CHIRA
OUTFLOW INTO SAN BARTOLO
Q = 1.0 m³/sec
OUTFLOW INTO SAN BARTOLO S.T.P.
Q = 2.5 m³/sec
PAMPA SAN BARTOLO
O C E A N O   P A C I F I C O
Plan | Phase I | Phase II
D I | 1.5 m³/sec | 2.5 m³/sec
LEGEND
Existing Intercepter
Planned Intercepter (Phase I)
Planned Intercepter (Phase II)
Gravity Main
Inverted Siphon
Force Main
Open Channel
Intake Structure
Diversion & Junction Structures
Outfall
Pumping Station
Treatment Works
V = 1/1,000
H = 1/50,000
TOMAS MARZANO
SAN JUAN DE MIRAFLORES
LOMO DE CORVINA
PACHACAMAC ZONA ARQUEOLOGICA
VALLE DE LURIN
PAMPA SAN BARTOLO
DIAMETER , LENGTH
TYPE OF FLOW
ø1350 L: 3,350m — Inverted Siphon
ø1350 L: 3,675m — Gravity
ø1200 L: 2,500m — Gravity
ø1100 L: 3,600m — Inverted Siphon
ø1100 L: 2,250m — Inverted Siphon
ø1100 L: 2,750m — Inverted Siphon
ø1200 L: 4,105m — Gravity
ø1100 L: 2,500m — Inverted Siphon
U 1700 × 850 L: 6,500m — Gravity
Q : 1.5 m³/sec V : 1.18 m/sec S : 0.8 ‰
Q : 1.5 m³/sec V : 1.23 m/sec S : 1.1 ‰
Q : 1.0 m³/sec V : 1.03 m/sec S : 0.9 ‰
Q : 1.0 m³/sec V : 1.17 m/sec S : 1.0 ‰
Q : 1.0 m³/sec V : 1.17 m/sec S : 1.0 ‰
Q : 1.0 m³/sec V : 1.17 m/sec S : 1.0 ‰
Q : 1.0 m³/sec V : 0.97 m/sec S : 0.8 ‰
Q : 1.0 m³/sec V : 1.17 m/sec S : 1.0 ‰
Q : 1.0 m³/sec V : 0.92 m/sec S : 0.5 ‰
FIGURE 7-13
SEDAPAL
Servicio de Agua Potable y Alcantarillado de Lima
FEASIBILITY STUDY ON IMPROVEMENT OF SEWERAGE SYSTEM IN SOUTHERN PART OF LIMA
TITLE: ALTERNATIVE - D PLAN DI LAYOUT PLAN LONGITUDINAL SECTION
SCALE: 1 / 50,000 DATE: '90 DRAWING No.:
JAPAN INTERNATIONAL COOPERATION AGENCY
DI



Plan	Phase I	Phase II
D2	1.0 m³/sec	3.0 m³/sec

FIGURE 7-14


INTAKE AT CIRCUNVALACION
Q = 1.0 m³/sec
GRIT CHAMBER
INTAKE AT VILLA MARIA DEL TRIUNFO
Q = 1.0 m³/sec
SEDAPAL
INTAKE AT SURCO
Q = 2.0 m³/sec
LA TABLADA
OUTFLOW INTO VILLA SALVADOR S.T.P.
Q = 0.5 m³/sec
OUTFLOW INTO PACIFIC OCEAN
Q = 0.5 m³/sec
Site of Treatment Plant (SEDAPAL)
Q = 2.5 m³/s
PUNTA LA CHIRA
OUTFLOW INTO SAN BARTOLO S.T.P.
Q = 1.5 m³/sec
OUTFLOW INTO SAN BARTOLO S.T.P.
Q = 2.0 m³/sec
PAMPA SAN BARTOLO
OCEANO PACIFICO
Plan | Phase I | Phase II
E1 | 2.0 m³/sec | 2.0 m³/sec
E2 | 1.0 m³/sec | 3.0 m³/sec
THIS PLAN SHOWS THE CASE OF PLAN E1.
LEGEND
Existing Intercepter
Planned Intercepter (Phase I)
Planned Intercepter (Phase II)
Gravity Main
Inverted Siphon
Force Main
Open Channel
Intake Structure
Diversion & Junction Structures
Outfall
Pumping Station
Treatment Works
Phase I
Phase II
V = 1/1,000
H = 1/50,000
AV. CIRCUNVALACION
AV. AGUSTIN LOZANO
ROSA TIRADO
AV. MIGUEL IGLESIAS
AV. MICAELA BASTIDAS
AV. PASTOR SEVILLA
ARENAL DE PACHACAMAC
VALLE DE LURIN
FABRICA DE EXPLOSIVOS (LINDERO PARTE ALTA)
DIAMETER, LENGTH TYPE OF FLOW
Inverted Siphon No.1
Gravity Flow No.2
Inverted Siphon No.2
Gravity Flow No.3
Inverted Siphon No.3
Gravity Flow No.4
FLOW CAPACITY
* For Phase I. Refer to the layout plan for Phase II.
FIGURE 7-15
SEDAPAL
Servicio de Agua Potable y Alcantarillado de Lima
FEASIBILITY STUDY ON IMPROVEMENT OF SEWERAGE SYSTEM IN SOUTHERN PART OF LIMA
TITLE: ALTERNATIVE - E
PLAN E1
LAYOUT PLAN
LONGITUDINAL SECTION
JAPAN INTERNATIONAL COOPERATION AGENCY

Hazen - Wiliams' Formula: $Q = A \cdot V$

$$V = 0.35464\ C D^{0.63} S^{0.54}$$

where;

Q: Quantity of flow (m^3/sec)

A: Flow area (m^2)

V: Flow velocity (m/sec)

C: Coefficient of roughness 130

D: Diameter (m)

S: Hydraulic gradient

Note : SEDAPAL uses the Manning's Formula for open channel, and the Hazen-Wiliams' Formula for pipeline.

(3) Flow velocity

The following velocity range is basically applied based on the planned sewage flow:

a) Gravity flow

Minimum 0.6 m/sec

Maximum 3.0 m/sec

b) Pressurized flow

Minimum 1.0 m/sec

Maximum 2.0 m/sec

(4) Pipe Materials

Pipe materials planned for alternatives are as follows (refer to APPENDIX 16):

a) Gravity flow section:

External pressure pipe (Reinforced concrete pipe)

b) Inverted siphon:

Internal pressure pipe (Prestressed concrete pipe, and Ductile cast iron pipe)

c) Pumping flow section:

Ductile cast iron pipe

d) Open channel

Open channel with concrete lining on both sides and bottom.

(5) Inlet chamber of inverted siphon

The inverted siphon will be made up with an inlet chamber at the upstream side of the pipe, placing screens and grit chambers.

(6) Manhole

The manhole will be installed in places where there are changes in pipe diameter, unevenness of invert level, pipe junctions, as well as in places deemed necessary for maintenance. A pressure type manhole will be installed where necessary.

(7) Blow-off or Drain

Blow-off valve will be installed at appropriate interval in long-span of inverted siphon pipes. Drain pipe will be connected to the nearest existing sewer if possible. However, along the coast line and at the Lurin river, the drain pipes are connected to sludge drying basin to prevent water pollution.

(8) Air valve

Air valves will be provided at the places deemed necessary along the inverted siphon section.

(9) Materials of pipe fittings

Pipe fittings and equipment for the pipeline with high inner pressure are made of cast iron or steel.

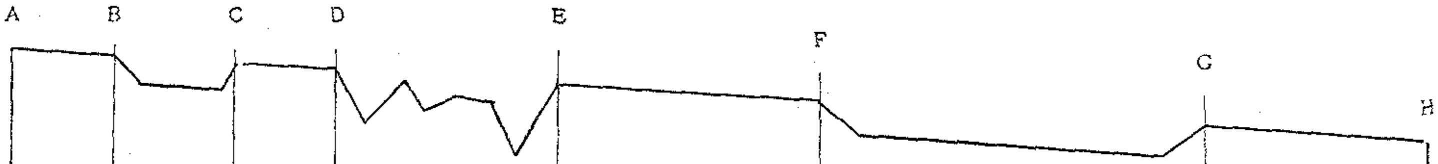
(10) Hydraulic calculation

As an example, outline and hydraulic calculations for alternative plan E_1 are presented in FIGUREs 7-16 and 7-17 for Phase I and Phase II, respectively. The figures likewise present design values for the planned sewage flow, maximum capacity, and minimum flow for allowable minimum flow velocity. Allowable minimum flow velocity is set as follows:

Gravity flow pipe:	0.6 m/s
Gravity flow open channel:	0.6 m/s
Inverted siphon:	0.8 m/s

Rough sketches of various structures necessary for the transmission facility are shown in APPENDIX 18.

High Elevation Transmission Line



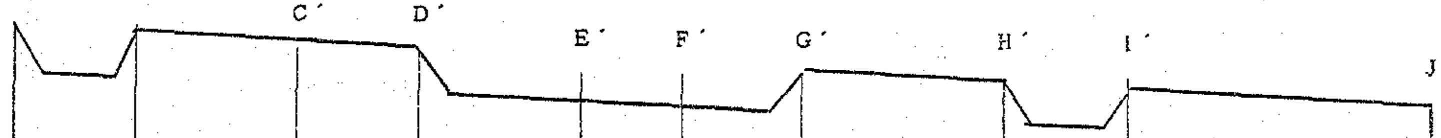
HYDRAULIC CALCULATION

		A–B	B–C	C–D	D–E	E–F	F–G	G–H	
		Q =1m³/S	Q =1m³/S	Q =1m³/S	Q =2m³/S	Q =1.5m³/S	Q =1.5m³/S	Q =1.5m³/S	GF : Gravity Flow IS : Inverted Siphon
		150 m	4,090 m	1,590 m	5,150 m	5,370 m	9,790 m	3,500 m	ΣL = 29,640 m
		φ800	φ1,000	φ1,100	φ1,400	φ1,350	φ1,200	1,900 x 950	
		GF	IS	GF	IS	GF	IS	GF	
GROUND CONDITION	GROUND ELEVATION	146.31	142.71	133.58	130.95	120.99	114.87	100.00	98.25
	ELEVATION BALANCE h (m)	3.60	9.12	2.63	9.96	6.12	14.87	1.75	Σ = 48.05 m
	GRADIENT (‰) h/L	24.00	2.22	1.65	1.93	1.13	1.5	0.5	
PLANNED SEWAGE AMOUNT	Qp(m³/S)	M 1.00	HW 1.00	M 1.00	HW 2.00	M 1.50	HW 1.50	M 1.50	HW: Hazen-Williams c=130
	V (m/S) I (‰)	2.63 8.0	1.27 2.2	1.41 1.5	1.30 1.9	1.38 1.1	1.33 1.5	0.99 0.5	M: Manning n=0.013
	Hf (m)	1.20	8.99	2.39	9.78	5.90	14.68	1.75	Σ = 44.69 m
	Hf<h (check)	OK	OK	OK	OK	OK	OK	OK	
MAXIMUM CAPACITY	Qm(m³/S)	1.18	1.32	1.19	2.55	1.77	1.75	1.80	
	V (m/S) I (‰)	2.35 8.0	1.69 2.2	1.26 1.5	1.66 1.9	1.23 1.1	1.54 1.5	1.00 0.5	
	Qm/Qp	1.18	1.32	1.19	1.27	1.18	1.17	1.20	
Q for MIN. VELOCITY	Q (m³/S)	0.006	0.625	0.045	1.228	0.070	0.909	0.327	
	V (m/S)	0.6	0.8	0.6	0.8	0.6	0.8	0.6	

FIGURE 7-16 HYDRAULIC CALCULATION OF TRANSMISSION LINE (PLAN E, PHASE I)

Low Elevation Transmission Line

A´ B´ C´ D´ E´ F´ G´ H´ I´ J´



Hydraulic Calculation

		A´–B´	B´–C´	C´–D´	D´–E´	E´–F´	F´–G´	G´–H´	H´–I´	I´–J´	
		Q = 2 m³/S	Q = 2 m³/S	Q = 2 m³/S	Q = 2 m³/S	Q=2 m³/S	Q = 2 m³/S	Q = 2 m³/S	Q = 2 m³/S	Q = 2 m³/S	GF : Gravity Flow IS : Inverted Siphon
		3,350 m	3,680 m	3,000 m	3,600 m	2,250 m (8,600 m)	2,750 m	4,110 m	2,500 m	6,500 m	ΣL = 31,740 m
		φ1400	φ1400	φ1500	φ1400	φ1400	φ1400	φ1500	φ1400	2,100 x 1,050	
		IS	GF	GF	IS	IS	IS	GF	IS	GF	
GROUND CONDITION	GROUND ELEVATION	89.08	83.13	77.34	74.37	2.30	1.80	62.09	56.07	52.20	48.95
	ELEVATION BALANCE h (m)	5.95	5.79	2.97		(12.28)		6.02	3.87	3.25	Σ= 40.13 m
	GRADIENT (‰) h/L	1.77	1.57	0.99		(1.43)		1.46	1.54	0.50	
PLANNED SEWAGE AMOUNT	Qp(m³/S)	HW 2.00	M 2.00	M 2.00	HW 2.00	HW 2.00	HW 2.00	M 2.00	HW 2.00	M 2.00	HW: Hazen-Williams c=130
	V (m/S) I (‰)	1.29 1.2	1.29 1.4	1.13 0.9	1.29 1.4	1.29 1.4	1.29 1.4	1.36 0.9	1.30 1.5	1.04 0.5	M: Manning n=0.013
	H_f (m)	4.02	5.15	2.70		(12.04)		3.69	3.75	3.25	Σ= 34.60 m
	H_f<h (check)	OK	OK	OK		OK		OK	OK	OK	
MAXIMUM CAPACITY	Qm(m³/S)	2.32	2.20	2.12	2.52	2.52	2.52	2.12	2.62	2.35	
	V (m/S) I (‰)	1.51 1.2	1.43 1.4	1.20 0.9	1.64 1.4	1.64 1.4	1.64 1.4	1.20 0.9	1.70 1.5	1.07 0.5	
	Qm/Qp	1.16	1.10	1.06	1.26	1.26	1.26	1.06	1.31	1.18	
Q for MIN. VELOCITY	Q (m³/S)	1.229	0.052	0.095	1.229	1.229	1.229	0.095	1.233	0.349	
	V (m/S)	0.8	0.6	0.6	0.8	0.8	0.8	0.6	0.8	0.6	

FIGURE 7-17 HYDRAULIC CALCULATION OF TRANSMISSION LINE (PLAN E₁, PHASE II)

7.4 Grit Chamber and Pumping Facility

7.4.1 Grit Chamber

(1) Basic Design Concept

a. In the Grit chamber, inorganic solids and suspended coarse matters in the raw sewage are removed in order to prevent the sedimentation of sand and erosion or clogging in transmission pipeline, pumps and treatment plant, for smooth operation and maintenance works. Grit Chamber, in principle, is located at the diversion point and upstream of inverted siphon, transmission pump or treatment plant.

b. Sedimented sand in the Grit Chamber is removed by manpower periodically after drainage of chamber. For this reason, more than two units of Grit Chamber are generally provided.

c. Hand rake screen is provided in the Grit Chamber to remove the suspended coarse matters.

(2) Design Criteria

Type	: Plug-Flow Rectangular Type
Water Surface Load	: S1 = Approx. 1,800 m^3/m^2/day
Average Velocity	: V = Approx. 0.30 m/s
Detention Time	: T = 30 to 60 sec.

(3) Outline of Structure

Outline of Grit Chamber structure is as follows:

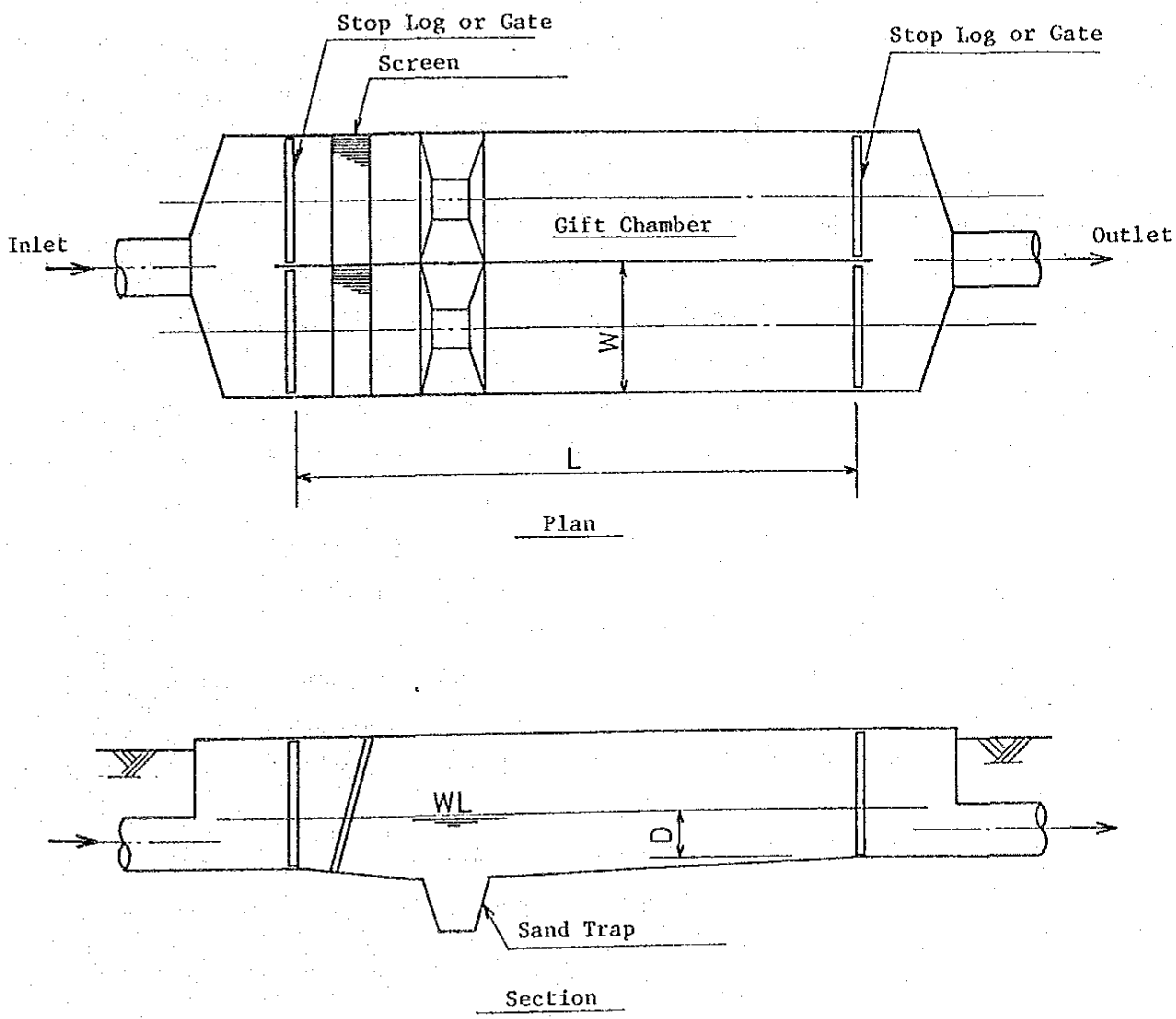


FIGURE 7-18 Outline of Grit Chamber Structure

(4) Dimension of Grit Chamber

Dimensions of Grit Chamber for each design flow of Alternatives are shown in TABLE 7-2.

TABLE 7-2 Dimension of Grit Chamber

Type	Design Flow (m^3/s)	Required Water Surface (m^2)	Dimension of Grit Chamber Width (mm)	Length (mm)	Depth (mm)	No. of Com-partment
I	0.19	0.19x86,400/1,800 = 9.12	1,000	9,500	600	1
	0.50	0.50x86,400/1,800 = 24	1,400	8,600	600	2
II	1.00	1.00x86,400/1,800 = 48	1,200	13,500	1,000	3
		1.00x86,400/3,600*= 24	1,500	9,500	1,300	2
	1.50	1.50x86,400/1,800 = 72	1,400	18,000	1,200	3
III	2.00	2.00x86,400/1,800 = 96	1,300	19,200	1,400	4
	2.50	2.50x86,400/1,800 =120	1,500	20,000	1,400	4
IV	3.00	3.00x86,400/1,800 =144	1,400	20,600	1,500	5
	3.50	3.50x86,400/1,800 =168	1,500	22,400	1,600	5
	4.00	4.00x86,400/1,800 =192	1,700	22,600	1,600	5

*: In case Water Surface Load is 3,600 $m^3/m^2/day$.

7.4.2 Pumping Facility

(1) Basic Design Concept

a. The number of pumps to be installed must be as few as possible with each pump unit having the same capacity and performance for easier operation and maintenance.

b. Installation number of pump based on "Guidelines and Explanation for Design of Sewage Facilities" (Japan Sewage Works Association, 1984) is as follows:

Design Flow (m^3/s)	Number of Pumps (set)
under 0.5	3 (incl. 1 stand-by)
0.5 to 1.5	3 to 5 (incl. 1 stand-by)
above 1.5	4 to 6 (incl. 1 stand-by)

c. Pump is of the vertical shaft centrifugal type because of less space, requirement and suitability to design flow and total head.

(2) Outline of Structure

Outline of pumping facility structure is as follows:

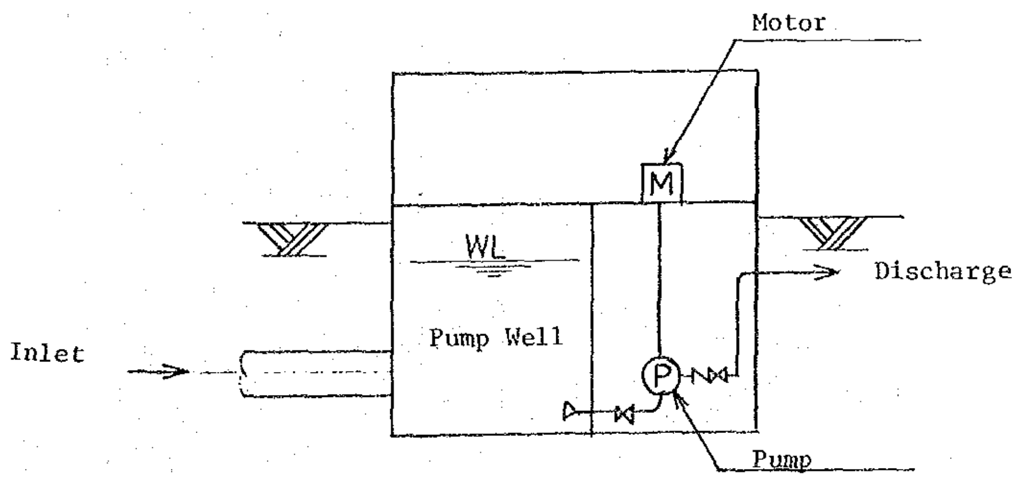


FIGURE 7-19 Outline of Pumping Facility Structure

(3) Capacity Calculations of Pump

a. Water Level Profile

Water level profile of the alternatives are as follows:

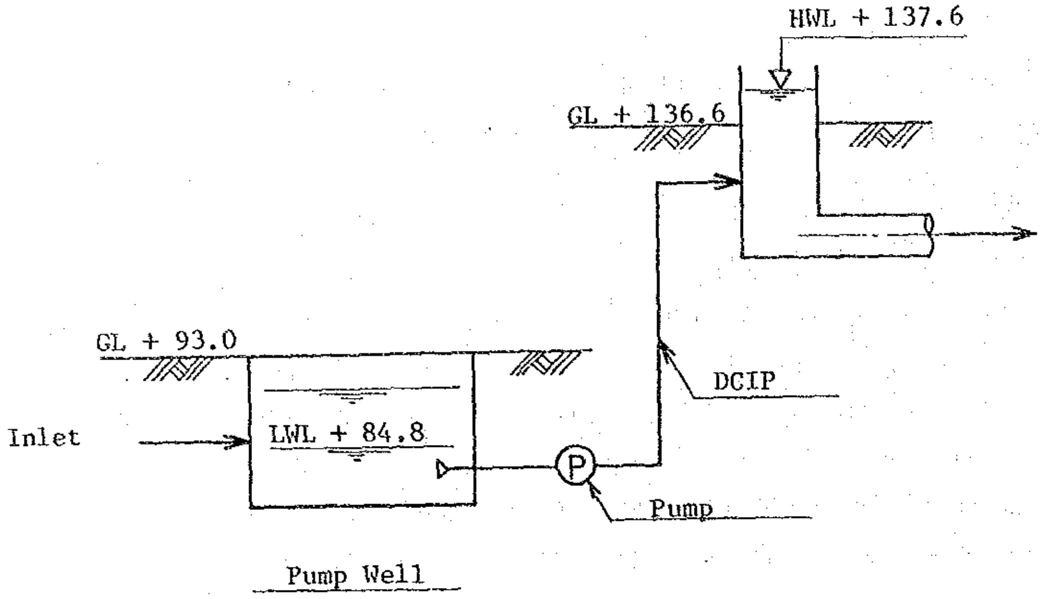


FIGURE 7-20 Water Level Profile of Pumping Facility (Alternatives A_1, A_2 and A_3)

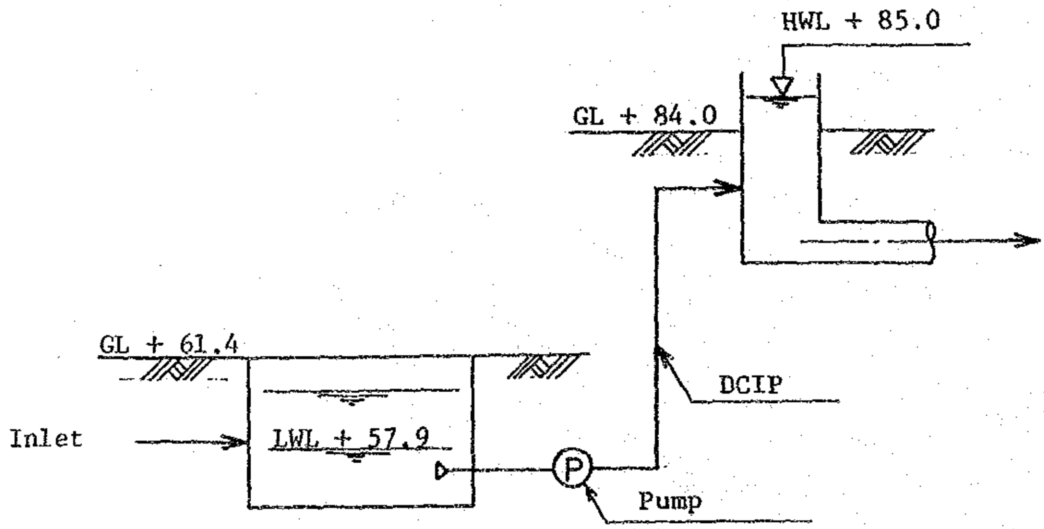


FIGURE 7-21 Water Level Profile of Pumping Facility (Alternatives C_1, C_2, D_1 and D_2)

b. Pump Capacity

Capacity calculations of pumps for each alternative are shown in TABLEs 7-3 and 7-4.

c. Application to Present Project

Because provisions must be made for power cuts, bypass discharge for emergency shall be considered.

TABLE 7－3 Capacity Calculation of Pump (Alternative A)

Item		Alternative	A_1 (Phase I)	A_2 (Phase I)	A_2 (Phase II)	A_3 (Phase II)
Design Flow		m^3/s	2.6 (156 m^3/min)	1.5 (90 m^3/min)	1.1 (66 m^3/min)	3.0 (1806 m^3/min)
Diameter of Pipeline		mm φ	1,350	1,100	900	1,500
Length of Pipeline		m	1,770	1,770	1,770	1,770
Number of Pump ※1		sets	6 (1)	5 (1)	4 (1)	6 (1)
Capacity of Pump per 1 set		m^3/min	156x1/5=31.2	90x1/4=22.5	66x1/3=22	180x1/5=36
Diameter of Pump		mm φ	500	450	450	600
Total Head	Actual Head	m	137.6-84.8=52.8	52.8	52.8	52.8
	Head Loss	m	5.9	5.8	7.9	4.9
	Total Head	m	58.7≒59	58.6≒59	60.7≒61	57.7≒58
Motor Shaft Power per 1 set		kW	$0.163x \frac{1.0x31.2x59}{0.8} =375$	$0.163x \frac{1.0x22.5x59}{0.79} =274$	$0.163x \frac{1.0x22x61}{0.79} =277$	$0.163x \frac{1.0x36x58}{0.81} =421$
Specification of Pump ※1			φ500x31.2m^3/minx59m x440kWx6set (1)	φ450x22.5m^3/minx59m x320kWx5set (1)	φ450x22m^3/minx61m x320kWx4set (1)	φ600x36m^3/minx58m x490kWx6set (1)

※1 : (1) means 1 stand-by set.

TABLE 7－4 Capacity Calculation of Pump (Alternative C or D)

Item / Alternative			C_1 or D_1 (Phase I)	C_2 or D_2 (Phase I)		
Design Flow		m^3/s	1.0 (60 m^3/min)	0.5 (30m^3/min)		
Diameter of Pipeline		mmϕ	ϕ900	ϕ600		
Length of Pipeline		m	900	900		
Number of Pump ※1		sets	4 (1)	3 (1)		
Capacity of Pump per 1 set		m^3/min	60x1/3=20	30x1/2=15		
Diameter of Pump		mmϕ	400	350		
Total Head	Actual Head	m	85.0-57.9=27.1	27.1		
	Head Loss	m	4.3	7.0		
	Total Head	m	31.4≒32	34.1≒35		
Motor Shaft Power per 1 set		kW	$0.163x \frac{1.0x20x32}{0.78} =134$	$0.163x \frac{1.0x15x35}{0.76} =113$		
Specification of Pump ※1			ϕ400x20m^3/minx32m x160kWx4set (1)	ϕ350x15m^3/minx35m x130kWx3set (1)		

※1 : (1) means 1 stand-by set.

7.5 Sewage Treatment Plant

7.5.1 General

Important items to be considered in the selection of treatment method are as follows:

a. Quantity and quality of sewage and their variation
b. Conditions in the areas of discharge and water use
c. Scale of treatment plant
d. Location and environment of treatment plant
e. Operation and maintenance organization
f. Operation and maintenance cost

Investigations must be made on these items in relation to the present Project and the most suitable treatment method selected accordingly.

7.5.2 Requirements of the System

(1) Design Sewage Flow

From investigations in Section 5.2, the discharge from Colector Surco at present is estimated at 5.0 m^3/s and is predicted to increase to around 6.5 m^3/s in the year 2000.

From the discussions in Chapter 6, the design flow of sewage treatment plant in each proposed site for several given alternatives are summarized in TABLE 7-5.

TABLE 7-5 Design Flow for Sewage Treatment Plant in Each Proposed Site

(unit : m^3/s)

Proposed Site		a. San Juan STP		b. San Juan		c. Villa El Salvador		e. & f. San Bartolo	
Alternatives		Ph-I	Ph-II	Ph-I	Ph-II	Ph-I	Ph-II	Ph-I	Ph-II
A	A1	-	-	-	-	0.5	-	3.5	-
	A2	-	-	-	-	0.5	-	1.5	2.0
	A3	-	-	-	-	0.5	-	0.5	3.0
B	B1	-	-	-	-	0.5	-	3.5	-
	B2	-	-	-	-	0.5	-	1.5	2.0
	B3	-	-	-	-	0.5	-	0.5	3.0
C	C1	0.5*1	-	1.0	-	0.83	-	-	1.67
	C2	-	-	1.0	-	0.83	-	-	2.17
	C3	-	-	-	0.5	0.83	-	-	2.67
	C3'	-	-	-	0.5	1.0	-	-	2.5
D	D1	0.5*1	-	1.0	-	-	-	-	2.5
	D2	-	-	1.0	-	-	-	-	3.0
E	E1	-	-	-	-	0.5	-	1.5	2.0
	E2	-	-	-	0.5	-	-	1.0	2.5

*1 0.5 m^3/s is the increase in quantity by reconstruction.

(2) Influent Sewage Quality

From the investigation in Section 5.5, Projected Sewage Quality, the influent sewage quality values are as follows;

BOD_5	:	250 mg/l
SS	:	250 mg/l

(3) Target Treated Water Quality

The primary purpose of this Project is to lower the contamination level of sea water in the coastal area of Metropolitan Lima. As a secondary purpose, it is aimed to reuse treated sewage for irrigation.

Target treated water quality must therefore be set in consideration of both purposes.

Based on the study in APPENDIX 19, target treated water quality is set as follows:

TABLE 7-6 Target Treated Water Quality

Site of Plan	Treatment Level	BOD5 (mg/l)	Fecal Coliforms (MPN/100ml)
On the west bank of Rio Lurin	3	35	1,000
In the San Bartolo	2	45	10,000

7.5.3 Treatment Method

(1) Basic Conception

As mentioned in APPENDIX 19, two main factors should be taken into account in this Project: namely, the land area and the operation and maintenance problem.

For the purpose of selecting alternatives, following aspects shall be considered in the evaluation :

<u>Land Requirement</u> <u>Operation and Maintenance</u>

Large ....Easy, low cost

Small ....Slightly easy, slightly lower cost

(2) Treatment Method

Study on treatment method is discussed in detail in APPENDIX 19. From investigations on available area and required treated water quality of each proposed site, and operation and maintenance problems, treatment method in each proposed site is decided as follows;

Proposed Site	Treatment Method
a. San Juan STP	Aerated Lagoon (AL)
b. San Juan	Aerated Lagoon (AL)
c. Villa el Salvador	Aerated Lagoon (AL)
e & f. San Bartolo	Waste Stabilization Pond (WSP)

7.5.4 Design Criteria

Study on Design Criteria for each Treatment Method is discussed in detail in APPENDIX 20. Design Criteria on Waste Stabilization Pond and Aerated Lagoon (Dual Power system) is given in TABLEs 7-7 and 7-8.

TABLE 7-7 Design Criteria of Waste Stabilization Ponds

Parameter	Symbol	Unit	Formula or Value	Application
Primary Facultative Pond				
• Water Temperature	Tw	°C	$T_w = 8.49 - 0.82\ T_a$	$T_a = 15\ °C$ $T_w = 8.49 + 0.82 \times 15 = 20.8\ °C$
• BOD_5 Areal Loading	Li1	kg-BOD/ha/d	under 400 $L_{i1} = 357.4 \times 1.085^{(T_w-20)}$	$L_{i1} = 357.4 \times 1.085^{(20.8-20)}$ $= 382$
• Water Depth	D1	m	1.3 - 1.6	1.5
• BOD_5 Removal Rate	R1	%	65 - 75	70
Secondary Facultative Pond				
• BOD_5 Areal Loading	Li2	kg-BOD/ha/d	40 - 210	200
• Water Depth	D2	m	1.3 - 1.6	1.5
• BOD_5 Removal Rate	R2	%	30 - 40	35

TABLE 7-8 Design Criteria for Aerated Lagoon (Dual Power Aeration System)

Parameter	Symbol	Unit	Formula or Value	Application
Complete Mixing Aerated Lagoon				
. Detention Time	t*c	day	1.5 - 2.0	2.0
. Water Depth	Dc	m	3.0 - 4.0	3.0
. Number of Lagoon	Nc	-	1	1
. Oxygen Requirement	Ro	kg/hr	$Ro = 6.24 \times 10^{-5} \times Q.Li$	Same as left
. Power Requirement for Mixing	pc	w/m^3	$pc >= 6w/m^3$	$6w/m^3$
Facultative Aerated Lagoon				
. Detention Time for One-Cell	t*f	day	0.5 - 1.0	0.67
. Water Depth	Df	m	3.0 - 4.0	3.0
. Power Requirement for Partially Mixing	pf	w/m^3	$pf >= 1\ w/m^3$	$1.0-1.5w/m^3$
. Number of Lagoon	nf	-	1 - 3 (series)	3 cells
Sedimentation Ponds				
. Detention Time	t*s	day	1 - 2	1

7.5.5 Outline of Facility

Detailed capacity calculation for sewage treatment plant on each proposed site is given in APPENDICES 13 and 14. Outline of each facility is presented in this section.

(1) Possible Treatment Capacity in Proposed Site (a) San Juan STP.

a. Basic Concept for Reconstruction

Out of the two-series treatment consisting of Upper and Lower Battery facultative ponds, treatment method of Upper Battery will be changed to Aerated Lagoon system in order to increase treatment capacity.

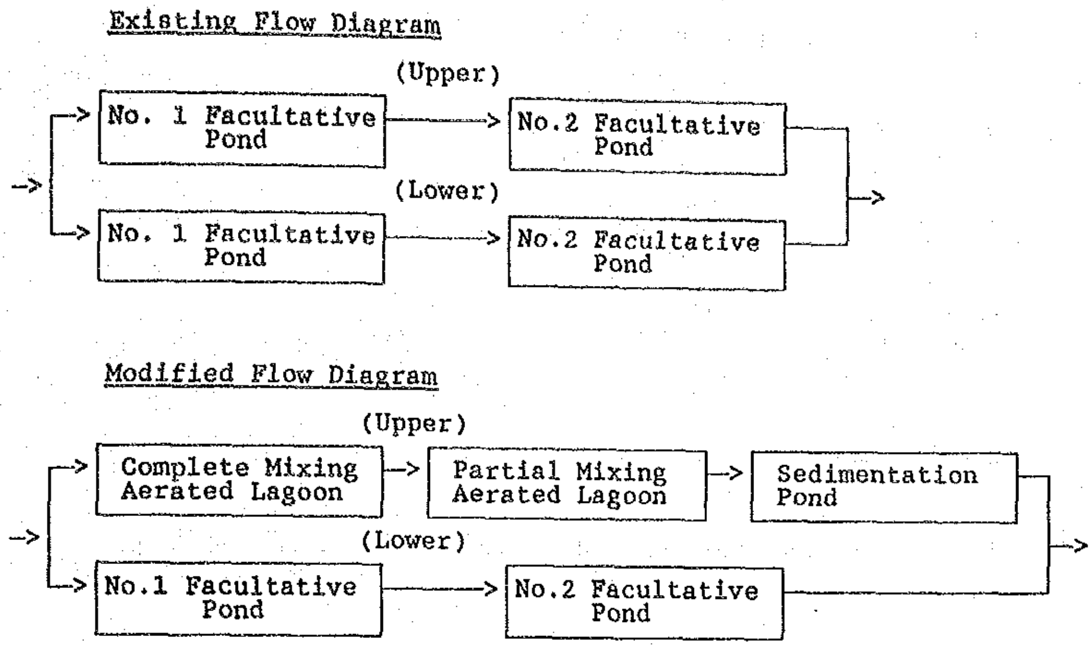


FIGURE 7-22 Flow Diagram of San Juan STP

b. Total treatment capacity in San Juan Sewage Treatment Plant after reconstruction

	Treatment Method	Possible Treatment Capacity	Total Treatment Capacity
Upper Battery (Reconstruction)	Aerated Lagoon	0.63 m^3/s	
			0.75 m^3/s
Lower Battery (Existing)	Facultative Pond	21,600 m^3/day x 1/2 = 10,800 m^3/day = approx. 0.12m^3/s	(64,800 m^3/day)

Present treatment sewage flow in San Juan Treatment Plant is around 0.25 m^3/s. Therefore, increase in possible total treatment capacity is:

$$0.75 - 0.25 = 0.5\ m^3/s$$

(2) Aerated Lagoon in Proposed Site (b) and (c)

a. Planned Treated Water Quality

	BOD	SS
Removal Rate	88 %	76 %
Water Quality	30 mg/l	60 mg/l

b. Flow Diagram

Flow diagram is shown in FIGURE 7-23.

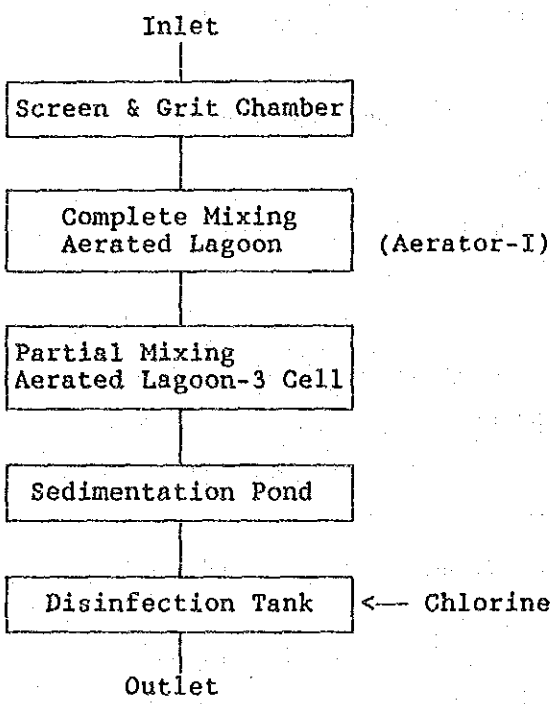


FIGURE 7-23 Flow Diagram for Aerated Lagoon (Dual Power) System

c. Outline of Aerated Lagoon System on Proposed Site (b) and (c)

Outline of facility for each design sewage flow is given in TABLE 7-9.

TABLE 7-9 Outline of Aerated Lagoon System on Proposed Site (b) and (c)

Design Flow Item	Qd avg = 0.5 m^3/s = around 43,200 m^3/day	Qd avg = 0.83 m^3/s = around 71,700 m^3/day	Qd avg = 1.0 m^3/s = around 86,400 m^3/s
1. Completely Mixing Aerated Lagoon			
- Dimension	100m x 48m x 3.0 mD x 14,400m^3 x 6 basins	105m x 45m x 3.0mD x 14,490m^3 x 10 basins	100m x 48m x 3.0mD x 14,400m^3 x 12 basins
- Detention Time	t^*c= 2.0 days	t^*c= 2.0 days	t^*c= 2.0 days
- Aerator-I	30 kW x 12 sets 18.5 kW x 12 sets (Total 582 kW)	30 kW x 20 sets 18.5 kW x 20 sets (Total 970 kW)	30 kW x 24 sets 18.5 kW x 24 sets (Total 1,164 kW)
2. Partial Mixing Aerated Lagoon			
- Dimension	100m x 33m x 3.0 mD x 9,900m^3 x 9 basins (3 cell x 3 series)	71m x 46m x 3.0mD x 9,798m^3 x 15 basins (3 cell x 5 series)	100m x 33m x 3.0mD x 9,900m^3 x 18 basins (3 cell x 6 Series)
- Detention Time	t^*c= 2.1 days	t^*c= 2.0 days	t^*p= 2.1 days
- Aerator-II	5.5 kW x 18 sets (Total 99 kW)	5.5 kW x 30 sets (Total 165 kW)	5.5 kW x 36 sets (Total 198 kW)
3. Chlorinator			
- Feeding rate	3.0 mg/l	3.0 mg/l	3.0 mg/l
- Chlorinator	10 kg-cl/hr x 2 sets (1 set-standby)	10 kg-cl/hr x 2 sets (1 set-standby)	15 kg-cl/hr x 2 sets (1 set-standby)

(3) Waste Stabilization Pond in Proposed Site (e) and (f) (San Bartolo)

a. Planned Treated Water Quality

BOD5 removal rate : 80 %

Effluent BOD5 : Approx. 49 mg/l

b. Flow Chart

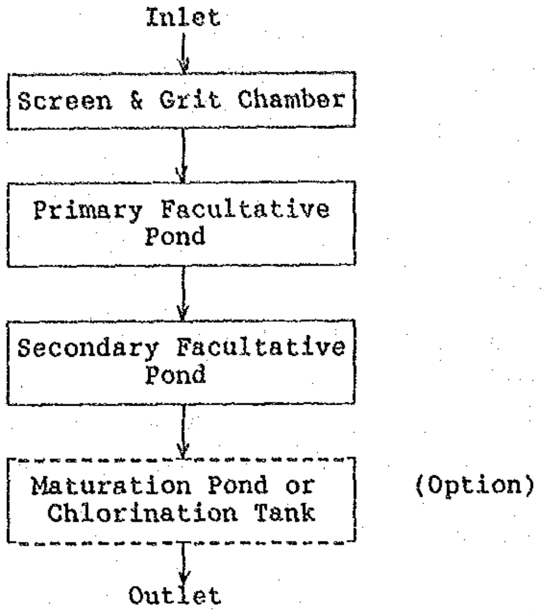


FIGURE 7-24 Flow Diagram for Stabilization Pond System

c. Outline of Waste Stabilization Pond System on Proposed Site (e) and (f)

Outline of facility for each design sewage flow is shown in TABLE 7-10.

TABLE 7-10 Outline of Waste Stabilization Pond System on Proposed Site (e) and (f)

Design Flow		Q (m^3/S)	0.5	1.5	1.67	2.0	2.17	2.5	2.67	3.0	3.5
Primary Facultative Pond	Lil	(kg-BOD/day)	10,800	32,400	36,072	43,200	46,872	54,000	57,672	64,800	75,600
	Ap	(ha)	28.3	84.8	94.4	113.1	122.7	141.4	151.0	169.6	197.9
	Dp	(m)	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	Vp	(m^3)	424,500	1,272,000	1,416,000	1,697,000	1,840,500	2,121,000	2,265,000	2,544,000	2,969,000
	t^*_P	(days)	9.8	9.8	9.8	9.8	9.8	9.8	9.8	9.8	9.8
Secondary Facultative Pond	As	(ha)	16.2	48.6	54.1	64.8	70.3	81.0	86.5	97.2	113.4
	Ds	(m)	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	Vs	(m^3)	243,000	729,000	811.500	972.000	1,054,500	1,215,000	1,298,000	1,458,000	1,701,000
	t^*_s	(days)	5.6	5.6	5.6	5.6	5.6	5.6	5.6	5.6	5.6
Total Mid-Depth Water Surface Area: Ap + As (ha)			44.5	133.4	148.5	177.9	193.0	222.4	237.5	266.8	311.3
Total Detention Time: $t^*_P + t^*_s$ (days)			15.4	15.4	15.4	15.4	15.4	15.4	15.4	15.4	15.4

7.6 Evaluation of Alternatives

7.6.1 Construction Cost

Estimated construction costs of each alternative are summarized in TABLEs 7-11 and 7-12. For the estimation of construction cost, 30 % of direct construction cost were added as overhead expenses of treatment method (for details refer to APPENDIX 21).

Construction costs per cubic meter of sewage required for Phase I project of each alternative are summarized in TABLE 7-13.

TABLE 7-13 COMPARISON OF CONSTRUCTION COST

PLAN		CONSTRUCTION COST C (1,000US$)	UNIT COST C/Q (US$/$m^3$)	RATIO (%)	RANKING	REMARKS
A	A_1	88,971	257	100	2	***
	A_2	111,537	323	125	10	*
	A_3	103,447	299	116	7	*
B	B_1	77,704	225	87	1	***
	B_2	97,314	282	109	5	**
	B_3	94,780	274	107	3	***
C	C_1	124,770	361	140	14	*
	C_2	122,442	354	138	12	*
	C_3	105,739	306	119	8	*
	C_3'	122,539	355	138	12	*
D	D_1	118,832	344	134	11	*
	D_2	108,077	313	121	9	*
E	E_1	98,301	284	110	6	**
	E_2	95,905	278	108	4	**

NOTE:
1) Ratios are shown in comparison to the C/Q of plan A_1 as 100%.
2) *** Superior, ** Intermediate, * Inferior
3) Q: 4 m^3/s x 86,400 s/day = 345,600 m^3/day

As a result, plans B_1, A_1 and B_3 are judged as superior, with plans E_2, B_2 and E_1 next to them.

TABLE 7-11 CONSTRUCTION COST (PHASE I)

Unit : US$1,000, ¥1,000,000

Plan		Q'TY m³/s	Direct Cost							Indirect Cost			Total	
			Intake Facility	Pumping Facility		Conduit	Treatment Facility		Sub-total	Over-head	Others[3]	Sub-total	US$	¥ (Equivalent)
				C & A[1]	Equip[2]		C & A[1]	Equip[2]						
A	A_1	4.0	477	5,629	10,510	39,647	10,308	4,831	71,402	17,508	61	17,569	88,971	12,455
	A_2	2.0	283	4,051	7,564	23,877	5,759	4,831	46,365	10,748	61	10,809	57,174	8,004
	A_3	1.0	184	—	—	12,604	3,170	4,831	20,789	5,004	61	5,065	25,854	3,619
B	B_1	4.0	477	—	—	45,035	10,308	4,831	60,651	16,963	90	17,053	77,704	10,878
	B_2	2.0	283	—	—	27,113	5,759	4,831	37,986	10,163	90	10,253	48,239	6,753
	B_3	1.0	184	—	—	15,513	3,170	4,831	23,698	5,877	90	5,967	29,665	4,153
C	C_1	2.33	354	3,178	5,934	28,203	3,861	26,162	67,692	12,122	249	12,371	80,063	11,208
	C_2	1.83	304	2,100	3,920	27,722	2,849	20,670	57,565	10,998	249	11,247	68,812	9,633
	C_3	0.83	163	—	—	16,552	1,292	9,489	27,496	5,829	59	5,888	33,384	4,673
	C_3'	1.0	184	—	—	23,090	1,557	11,180	36,011	7,952	65	8,017	44,028	6,163
D	D_1	1.5	191	3,178	5,934	23,682	2,569	16,673	52,227	9,903	191	10,094	62,321	8,724
	D_2	1.0	141	2,100	3,920	16,914	1,557	11,180	35,812	6,892	191	7,083	42,895	6,005
E	E_1	2.0	283	—	—	27,201	5,494	4,831	37,809	11,338	61	11,399	49,208	6,889
	E_2	1.0	141	—	—	16,403	3,320	—	19,864	5,959	—	5,959	25,823	3,615

※1 C & A: Construction Cost for Civil and Architectural Works.
※2 Equip: Cost for Mechanical and Electrical Equipment.
※3 Others: Cost for Plantation.
※4 The Base Date of Cost Estimates is October 26, 1989.
※5 Exchange Rate: I/. 6,050.75 = US$ 1.00, US$ 1.00 = ¥ 140

TABLE 7-12 CONSTRUCTION COST (PHASE II)

Unit: US$1,000, ¥1,000,000

Plan		Q'TY m³/s	Direct Cost							Indirect Cost			Total		Grand Total (I+II)	
			Intake Facility	Pumping Facility		Conduit	Treatment Facility		Sub-total	Overhead	Others[3]	Sub-total	US$	¥ (Equivalent)	US $	¥ (Equivalent)
				C & A[1]	Equip[2]		C & A[1]	Equip[2]								
A	A_1	—	—	—	—	—	—	—	—	—	—	—	—	—	88,971	12,455
	A_2	2.0	283	3,366	6,283	27,143	5,976	—	43,051	11,312	—	11,312	54,363	7,610	111,537	15,614
	A_3	3.0	255	6,133	11,449	36,128	7,968	—	61,933	15,660	—	15,660	77,593	10,863	103,447	14,482
B	B_1	—	—	—	—	—	—	—	—	—	—	—	—	—	77,704	10,878
	B_2	2.0	226	—	—	31,548	5,976	—	37,750	11,325	—	11,325	49,075	6,870	97,314	13,623
	B_3	3.0	255	—	—	41,866	7,968	—	50,089	15,026	—	15,026	65,115	9,116	94,780	13,269
C	C_1	1.67	212	—	—	28,900	5,278	—	34,390	10,317	—	10,317	44,707	6,258	124,770	17,466
	C_2	2.17	246	—	—	34,534	6,474	—	41,254	12,376	—	12,376	53,630	7,508	122,442	17,141
	C_3	3.17	269	—	—	42,889	8,548	4,831	56,537	15,728	90	15,818	72,355	10,129	105,739	14,802
	C_3'	3.0	255	—	—	48,103	8,083	4,831	61,272	17,149	90	17,239	78,511	10,991	122,539	17,154
D	D_1	2.5	248	—	—	36,151	7,071	—	43,470	13,041	—	13,041	56,511	7,911	118,832	16,635
	D_2	3.0	255	—	—	41,917	7,968	—	50,140	15,042	—	15,042	65,182	9,125	108,077	15,130
E	E_1	2.0	226	—	—	31,563	5,976	—	37,765	11,328	—	11,328	49,093	6,873	98,301	13,762
	E_2	3.0	255	—	—	41,619	8,083	4,831	54,788	15,204	90	15,294	70,082	9,811	95,905	13,427

※1 C & A: Construction Cost for Civil and Architectural Works
※2 Equip: Cost for Mechanical and Electrical Equipment.
※3 Others: Cost for Plantation.
※4 The Base Date of Cost Estimates is October 26, 1989.
※5 Exchange Rate: I/. 6,050.75 = US$ 1.00, US$ 1.00 = ¥ 140

7.6.2 Operation and Maintenance Cost

The operation and maintenance costs of each alternative are summarized in TABLE 7-14.

Bases of estimation are also presented in TABLEs 7-15 to 7-21.

O&M costs per cubic meter of sewage required for facilities in Phase I of each alternative are shown in TABLE 7-22.

TABLE 7-22 COMPARISON OF O&M COST

PLAN		O&M COST Cm (US$/year)	UNIT COST Cm/Q (10^{-3}US$/m^3)	RATIO (%)	RANKING	REMARKS
A	A_1	562,250	4.46	100	11	*
	A_2	594,115	4.71	106	12	*
	A_3	616,752	4.89	110	14	*
B	B_1	140,345	1.11	25	1	***
	B_2	155,669	1.23	28	3	**
	B_3	144,477	1.15	26	2	***
C	C_1	598,050	4.74	106	13	*
	C_2	437,063	3.46	78	9	*
	C_3	293,577	2.33	52	7	*
	C_3'	326,781	2.59	58	8	*
D	D_1	446,782	3.54	79	10	*
	D_2	282,520	2.24	50	6	*
E	E_1	154,185	1.22	27	3	**
	E_2	152,923	1.21	27	3	**

NOTE:
1) O&M cost is estimated for direct cost only.
2) Ratios are shown in comparison to the Cm/Q of plan A_1 as 100%.
3) Q: $4m^3/s \times 86{,}400$ s/day x 365 days/year = 126,144,000 m^3/year

Therefore, plans B_1 and B_3 are judged to be superior, with plans B_2, E_1 and E_2 next to them.

TABLE 7-14 OPERATION & MAINTENANCE COST

Unit : US$ / year

Plan		PHASE I								PHASE II								Total		Remarks
		Q'ty	Conduit	Pumping Facility		Treatment Facility			*1 Sub-total	Q'ty	Conduit	Pumping Facility		Treatment Facility			*1 Sub-total	US$ /Year	1,000 ¥/year (Equiv.)	
		m³/s	Labor	Labor	Power	Labor	Power	Chemicals		m³/s	Labor	Labor	Power	Labor	Power	Chemicals				
A	A_1	4.0	3,084	11,232	410,625	48,048	89,261	(179,124)	562,250	—	—	—	—	—	—	—	—	562,250	78,715	
	A_2	2.0	2,048	8,736	240,024	32,572	89,261	(89,562)	372,641	2.0	2,048	7,485	181,989	29,952	—	(89,562)	221,474	594,115	83,176	
	A_3	1.0	1,368	—	—	18,096	89,261	(44,781)	108,725	3.0	2,104	11,232	460,995	33,696	—	(134,343)	508,027	616,752	86,345	
B	B_1	4.0	3,036	—	—	48,048	89,261	(179,124)	140,345	—	—	—	—	—	—	—	—	140,345	19,648	
	B_2	2.0	2,000	—	—	32,572	89,261	(89,562)	123,833	2.0	1,884	—	—	29,952	—	(89,562)	31,836	155,669	21,793	
	B_3	1.0	1,320	—	—	18,096	89,261	(44,781)	108,677	3.0	2,104	—	—	33,696	—	(134,343)	35,800	144,477	20,226	
C	C_1	2.33	2,244	6,240	88,038	27,456	445,802	*2 (115,759)	569,780	1.67	1,688	—	—	26,582	—	(74,784)	28,270	598,050	83,727	
	C_2	1.83	1,972	3,744	49,494	22,464	327,022	(81,949)	404,696	2.17	1,916	—	—	30,451	—	(97,175)	32,367	437,063	61,188	
	C_3	0.83	1,188	—	—	13,728	148,500	(37,168)	163,416	3.17	2,212	—	—	38,688	89,261	(119,566)	130,161	293,577	41,100	
	C_3'	1.0	1,464	—	—	13,728	178,522	(44,781)	193,714	3.0	2,248	—	—	41,558	89,261	(134,343)	133,067	326,781	45,749	
D	D_1	1.5	1,612	6,240	88,038	18,720	297,302	*2 (78,591)	411,912	2.5	2,048	—	—	32,822	—	(111,953)	34,870	446,782	62,549	
	D_2	1.0	1,232	3,744	49,494	13,728	178,522	(44,781)	246,720	3.0	2,104	—	—	33,696	—	(134,343)	35,800	282,520	39,552	
E	E_1	2.0	1,888	—	—	31,200	89,261	(89,562)	122,349	2.0	1,884	—	—	29,952	—	(89,562)	31,836	154,185	21,585	
	E_2	1.0	1,286	—	—	18,720	—	(44,781)	20,000	3.0	2,104	—	—	41,558	89,261	(134,343)	132,923	152,923	21,409	

Note : * 1 Cost for Chemicals (for Chlorine) is not included.
* 2 For total sewage amount of (a) San Juan S.T.P. ($Q = 0.75 m^3/s$)
* 3 Exchange Rate : I/. 6,050.75 = US$ 1.00, US$ 1.00 = ¥ 140

TABLE 7-15 OPERATION & MAINTENANCE COST FOR TRANSMISSION LINE (LABOR COST FOR PHASE I)

Plan		① Intake Facility (persons/year)	② Gravity Flow Section (persons/year)	③ Inverted Siphon Section (persons/year)	Total No. (persons)	Total Unit Cost US$/person	Total Amount US$/year	Remarks
A	A_1	(2.6m³/s×0.74+0.9×1.0+0.5×1.3) ×72 p/m³ = 243	(4.0m³/s×0.6) ×200 = 480	span p/span 4 × 12 = 48	771	4	3,084	
	A_2	(1.5×0.9+0.5×1.3) ×72 = 144	(2.0×0.8) ×200 = 320	4 × 12 = 48	512	4	2,048	
	A_3	(0.5×1.3×2) ×72 = 94	(1.0×1.0) ×200 = 200	4 × 12 = 48	342	4	1,368	
B	B_1	(2.6×0.7+0.9×1.0+0.5×1.3) ×72 = 243	(4.0×0.6) ×200 = 480	3 × 12 = 36	759	4	3,036	
	B_2	(1.5×0.9+0.5×1.3) ×72 = 144	(2.0×0.8) ×200 = 320	3 × 12 = 36	500	4	2,000	
	B_3	(0.5×1.3×2) ×72 = 94	(1.0×1.0) ×200 = 200	3 × 12 = 36	330	4	1,320	
C	C_1	(1.5×0.9+0.5×1.3+0.33×1.3) ×72 = 175	(2.33 ×0.75) ×200 = 350	3 × 12 = 36	561	4	2,244	
	C_2	(1.0×1.0+0.5×1.3+0.33×1.3) ×72 = 150	(1.83 ×0.84) ×200 = 307	3 × 12 = 36	493	4	1,972	
	C_3	(0.5×1.3+0.33×1.3) ×72 = 78	(0.83 ×1.1) ×200 = 183	3 × 12 = 36	297	4	1,188	
	C_3'	(0.5×1.3+0.17×1.3+0.33×1.3) ×72 = 94	(1.0×1.0) ×200 = 200	6 × 12 = 72	366	4	1,464	
D	D_1	(1.5×0.9) ×72 = 97	(1.5×0.9) ×200 = 270	3 × 12 = 36	403	4	1,612	
	D_2	(1.0×1.0) ×72 = 72	(1.0×1.0) ×200 = 200	3 × 12 = 36	308	4	1,232	
E	E_1	(1.0×1.0×2) ×72 = 144	(2.0×0.7) ×200 = 280	4 × 12 = 48	472	4	1,888	
	E_2	(1.0×1.0) ×72 = 72	(1.0×1.0) ×200 = 200	4 × 12 = 48	320	4	1,280	

① Intake Facility
Number of persons required for Intake facility per 1m³/s is:
4 persons /facility × 3 days × 6 times/ year = 72 persons /m³/s
In accordance with the scale of facility, following adjusting factors P are multiplied.
3 m³/s P= 0.6
2 〃 P= 0.8
1 〃 P= 1.0

② Gravity Flow Section
Assumption : Length:15km, Manhole span:100m,
No. of Manholes: 150,
Cleaning Frequency: every 3 years
No. of Spans to be cleaned per year : 150 ÷3 = 50 span/year
Required day for cleaning : 1 day/span.
No. of Laborers per span : 4 persons/span
Annual labor requirement : 4 person · day/span x 50 span/year x 1 day = 200 persons · day/year

③ Inverted Siphon
Necessary Working days per span : 3 days
No. of laborers per team : 4
Required No. of laborers per span is:
3×4 = 12 persons/span

TABLE 7-16 OPERATION & MAINTENANCE COST FOR TRANSMISSION LINE
(LABOR COST FOR PHASE Ⅱ)

Plan		① Intake Facility (persons / year)	② Gravity Flow Seciton (persons / year)	③ Inverted Siphon Section (persons / year)	Total No. (persons)	Total Unit Cost US$/person	Total Amount US$/year	Remarks
A	A_1	—	—	—	—	—	—	
	A_2	(1.1m³/s×1.0+0.9×1.0) ×72 persons/m³/s= 144	m³/s (2.0 ×0.8)×200 = 320	span person/s 4 × 12 = 48	512	4	2,048	
	A_3	(3.0 ×0.6) × 72 = 130	(3.0×0.6)× 200 = 360	3 × 12 = 36	526	4	2,104	
B	B_1	—	—	—	—	—	—	
	B_2	(2.0 ×0.8)× 72 = 115	(2.0×0.8)× 200 = 320	3 × 12 = 36	471	4	1,884	
	B_3	(3.0 ×0.6)× 72 = 130	(3.0×0.6)× 200 = 360	3 × 12 = 36	526	4	2,104	
C	C_1	(1.67×0.85) ×72= 102	(1.67 ×0.85) ×200 = 284	3 × 12 = 36	422	4	1,688	
	C_2	(2.17×0.75) ×72= 117	(2.17 ×0.75) ×200 = 326	3 × 12 = 36	479	4	1,916	
	C_3	(3.17×0.6)× 72 = 137	(3.17 ×0.6)×200 = 380	3 × 12 = 36	553	4	2,212	
	C_3'	(3.0 ×0.6)× 72 = 130	(3.0×0.6)× 200 = 360	6 × 12 = 72	562	4	2,248	
D	D_1	(2.5 ×0.7)× 72 = 126	(2.5×0.7)× 200 = 350	3 × 12 = 36	512	4	2,048	
	D_2	(3.0 ×0.6)× 72 = 130	(3.0×0.6)× 200 = 360	3 × 12 = 36	526	4	2,104	
E	E_1	(2.0 ×0.8)× 72 = 115	(2.0×0.8)× 200 = 320	3 × 12 = 36	471	4	1,884	
	E_2	(3.0 ×0.6)× 72 = 130	(3.0×0.6)× 200 = 360	3 × 12 = 36	526	4	2,104	

① Intake Facility
Number of persons required for Intake facility per 1m³/s is:
4 persons /facility × 3 days × 6 times/ year = 72 persons / m³/s
In accordance with the scale of facility, following adjusting factors P are multiplied.
3 m³/s P= 0.6
2 〃 P= 0.8
1 〃 P= 1.0
0.5 〃 P= 1.3

② Gravity Flow Section
Assumption : Length:15km, Manhole span:100m,
No. of Manholes: 150,
Cleaning Frequency: every 3 years
No. of Spans to be cleaned per year : 150 ÷3 = 50 span/year
Required day for cleaning : 1 day per span.
No. of Laborers per span : 4 persons/span
Annual labor requirement : 4 persons/span × 50 span/year × 1 day= 200 persons/year

③ Inverted Siphon
Necessary Working days per span : 3 days
No. of laborers per team : 4
Required No. of laborers per span is:
3×4=12 persons/span

TABLE 7-17 OPERATION & MAINTENANCE COST FOR PUMPING STATION (LABOR COST)

Plan		PHASE I				PHASE II				REMARKS
		Number of Persons *1	Operation Days *2	Unit Cost	Amount	Number of Persons *1	Operation Days *2	Unit Cost	Amount	
A	A_1	m^3/s persons/m^3/s persons 2.6 × 0.7 × 5 = 9	days 312	US$/person 4 /day	US$/year 11,232	——— persons	day/year —	US$/person —	US$/year —	
	A_2	1.5 × 0.9 × 5 = 7	312	4	8,736	1.1 × 1.0 × 5 = 6	312	4	7,485	
	A_3	———	—	—	—	3.0 × 0.6 × 5 = 9	312	4	11,232	
B	B_1	———	—	—	—	———	—	—	—	
	B_2	———	—	—	—	———	—	—	—	
	B_3	———	—	—	—	———	—	—	—	
C	C_1	1.0 × 1.0 × 5 = 5	312	4	6,240	———	—	—	—	
	C_2	0.5 × 1.3 × 5 = 3	312	4	3,744	———	—	—	—	
	C_3	———	—	—	—	———	—	—	—	
	C_3'	———	—	—	—	———	—	—	—	
D	D_1	1.0 × 1.0 × 5 = 5	312	4	6,240	———	—	—	—	
	D_2	0.5 × 1.3 × 5 = 3	312	4	3,744	———	—	—	—	
E	E_1	———	—	—	—	———	—	—	—	
	E_3	———	—	—	—	———	—	—	—	

Note : * 1 Number of Persons per $1 m^3/s$
Labor 3 + Elec. Tech. 1 + Mech. Tech. 1 = 5 persons/m^3/s
* 2 Operation Days
6 day/week × 52 week/year = 312 day/year

TABLE 7-18 OPERATION & MAINTENANCE COST FOR TREATMENT FACILITY (LABOR COST)

Plan		PHASE I No. of Persons					PHASE I Operation Days	PHASE I Unit Cost	PHASE I Amount	PHASE II No. of Persons		PHASE II Operation Days	PHASE II Unit Cost	PHASE II Amount	Remarks
		a	b	c	Total		days	$/person	$/year		persons	days	$/person	$/year	
A	A_1	c (0.5m³/s) 7 人	(3.5×0.6)×15			38.5	312	4	48,048	———		—	—	—	
A	A_2	7	(1.5×0.85) ×15			26.1	312	4	32,572	(2.0 ×0.8)×15 = 24.0		312	4	29,952	
A	A_3	7	(0.5×1.0)×15			14.5	312	4	18,096	(3.0 ×0.6)×15 = 27.0		312	4	33,696	
B	B_1	b (0.5m³/s) 7	(3.5×0.6)×15			38.5	312	4	48,048	———		—	—	—	
B	B_2	7	(1.5×0.85) ×15			26.1	312	4	32,572	(2.0 ×0.8)×15 = 24.0		312	4	29,952	
B	B_3	7	(0.5×1.0)×15			14.5	312	4	18,096	(3.0 ×0.6)×15 = 27.0		312	4	33,696	
C	C_1	1 1 4 1	2 → 1 2 → 1 6 1 → 0	2 → 1 2 → 1 6 1	3 3 14 2	person 22	312	4	27,456	(1.67×0.85) × 15 = 21.3		312	4	26,582	
C	C_2		2 → 1 2 → 1 6 1	2 → 1 2 → 1 6 1	2 2 12 2	18	312	4	22,464	(2.17×0.75) × 15 = 24.4		312	4	30,451	
C	C_3			2 2 6 1	2 2 6 1	11	312	4	13,728	b (0.5m³/s) 7 (2.67 ×0.6)×15 = 24.0	Total 31.0	312	4	38,688	
C	C_3'			2 2 6 1	2 2 6 1	11	312	4	13,728	7 (2.5 ×0.7)×15 = 26.3	33.3	312	4	41,558	
D	D_1	1 1 4 1	2 → 1 2 → 1 6 1 → 0		2 2 10 1	15	312	4	18,720	(2.5 ×0.7) × 11 = 19.3		312	4	32,822	
D	D_2		2 2 6 1		2 2 6 1	11	312	4	13,728	(3.0 ×0.6) × 11 = 19.8		312	4	33,696	
E	E_1	C(0.5m³/s) 7	(1.5 ×0.8)×15 = 18.0			25	312	4	31,200	(2.0 × 0.8) × 15 = 24.0		312	4	29,952	
E	E_2	(1.0 ×1.0)×15 = 15.0					312	4	18,720	b (0.5m³/s) 7 (2.5× 0.7) ×15 = 26.3	33.3	312	4	41,558	

Operation Days : 6 days/week × 52 week = 312 days/year

	Mechanical	Electrical	Civil	Water Quality	Total	Remarks
Stabilization Pond	persons —	persons —	persons 14	persons 1	persons 15	per 1.0m³/s
Aerated	2	2	6	1	11	per 1.0m³/s
Lagoon	1	1	4	1	7	per 0.5m³/s

TABLE 7-19 OPERATION & MAINTENANCE COST FOR PUMPING STATION (POWER COST)

Plan		PHASE I				PHASE II				Remarks
		Daily Electricity Consumption	Operation Days	Unit Cost	Amount	Daily Electricity Consumption	Operation Days	Unit Cost	Amount	
		kW × pumps × hrs. = kWH/day	day/year	US$/kWH	US$/year	kWH/day	day/year	US$/kWH	US$/year	
A	A_1	375 × 5 × 24 = 45,000	365	0.025	410,625	—	—	—	—	
	A_2	274 × 4 × 24 = 26,304	365	0.025	240,024	277 × 3 × 24 = 19,944	365	0.025	181,989	
	A_3	—	—	—	—	421 × 5 × 24 = 50,520	365	0.025	460,995	
B	B_1	—	—	—	—	—	—	—	—	
	B_2	—	—	—	—	—	—	—	—	
	B_3	—	—	—	—	—	—	—	—	
C	C_1	134 × 3 × 24 = 9,648	365	0.025	88,038	—	—	—	—	
	C_2	113 × 2 × 24 = 5,424	365	0.025	49,494	—	—	—	—	
	C_3	—	—	—	—	—	—	—	—	
	C_3'	—	—	—	—	—	—	—	—	
D	D_1	134 × 3 × 24 = 9,648	365	0.025	88,038	—	—	—	—	
	D_2	113 × 2 × 24 = 5,424	365	0.025	49,494	—	—	—	—	
E	E_1	—	—	—	—	—	—	—	—	
	E_2	—	—	—	—	—	—	—	—	

TABLE 7-20 OPERATION & MAINTENANCE COST FOR TREATMENT FACILITY

(POWER COST)

Plan		PHASE I				PHASE II				REMARKS
		Daily Electricity Consumption (kWH/day)	Operation Days (day/year)	Unit Cost (US$/kWH)	Amount (US$/year)	Daily Electricity Consumption (kWH/day)	Operation Days (day/year)	Unit Cost (US$/kWH)	Amount (US$/year)	
A	A_1	(d) 9,782	365	0.025	89,261	—	—	—	—	
	A_2	(d) 9,782	365	0.025	89,261	—	—	—	—	
	A_3	(d) 9,782	365	0.025	89,261	—	—	—	—	
B	B_1	(d) 9,782	365	0.025	89,261	—	—	—	—	
	B_2	(d) 9,782	365	0.025	89,261	—	—	—	—	
	B_3	(d) 9,782	365	0.025	89,261	—	—	—	—	
C	C_1	(a) 13,017 (b) 19,564 (c) 16,274 } 48,855	365	0.025	445,802	—	—	—	—	
	C_2	(b) 19,564 (c) 16,274 } 35,838	365	0.025	327,022	—	—	—	—	
	C_3	(c) 16,274	365	0.025	148,500	(d) 9,782	365	0.025	89,261	
	C_3'	(c) 19,564	365	0.025	178,522	(d) 9,782	365	0.025	89,261	
D	D_1	(a) 13,017 (b) 19,564 } 32,581	365	0.025	297,302	—	—	—	—	
	D_2	(b) 19,564 19,564	365	0.025	178,522	—	—	—	—	
E	E_1	(d) 9,782	365	0.025	89,261	—	—	—	—	
	E_2	—	—	—	—	(d) 9,782	365	0.025	89,261	

(a) $0.63m^3/s$ (b) $1.0m^3/s$ (c) $0.83m^3/s$ (d) $0.5m^3/s$

TABLE 7-21 OPERATION & MAINTENANCE COST FOR TREATMENT FACILITY
(CHEMICALS COST *1)

Plan		PHASE I				PHASE II				REMARKS
		Daily Chlorine Consumption *2	Operation Days	Unit Cost	Amount	Daily Chlorine Consumption	Operation Days	Unit Cost	Amount	
A	A_1	$4.0 m^3/s \times 86,400 \times 2ppm \times 10^{-3}$ = 691.2 kg/day	day/year 365	US$/kg 0.71	US$/year 179,124	———	day/year —	US$/kg —	US$/year —	
	A_2	$2.0 \times 86,400 \times 2 \times 10^{-3}$ = 345.6	365	0.71	89,562	$2.0 m^3/s \times 86,400 \times 2ppm \times 10^{-3}$ = 345.6 kg/day	365	0.71	89,562	
	A_3	$1.0 \times 86,400 \times 2 \times 10^{-3}$ = 172.8	365	0.71	44,781	$3.0 \times 86,400 \times 2 \times 10^{-3}$ = 518.4	365	0.71	134,343	
B	B_1	$4.0 \times 86,400 \times 2 \times 10^{-3}$ = 691.2	365	0.71	179,124	———	—	—	—	
	B_2	$2.0 \times 86,400 \times 2 \times 10^{-3}$ = 345.6	365	0.71	89,562	$2.0 \times 86,400 \times 2 \times 10^{-3}$ = 345.6	365	0.71	89,562	
	B_3	$1.0 \times 86,400 \times 2 \times 10^{-3}$ = 172.8	365	0.71	44,781	$3.0 \times 86,400 \times 2 \times 10^{-3}$ = 518.4	365	0.71	134,343	
C	C_1	$2.585^{*3} \times 86,400 \times 2 \times 10^{-3}$ = 446.7	365	0.71	115,759	$1.67 \times 86,400 \times 2 \times 10^{-3}$ = 288.6	365	0.71	74,784	
	C_2	$1.83 \times 86,400 \times 2 \times 10^{-3}$ = 316.2	365	0.71	81,949	$2.17 \times 86,400 \times 2 \times 10^{-3}$ = 375.0	365	0.71	97,175	
	C_3	$0.83 \times 86,400 \times 2 \times 10^{-3}$ = 143.4	365	0.71	37,168	$2.67 \times 86,400 \times 2 \times 10^{-3}$ = 461.4	365	0.71	119,566	
	C_3'	$1.0 \times 86,400 \times 2 \times 10^{-3}$ = 172.8	365	0.71	44,781	$3.0 \times 86,400 \times 2 \times 10^{-3}$ = 518.4	365	0.71	134,343	
D	D_1	$1.755^{*3} \times 86,400 \times 2 \times 10^{-3}$ = 303.3	365	0.71	78,591	$2.5 \times 86,400 \times 2 \times 10^{-3}$ = 432	365	0.71	111,953	
	D_2	$1.0 \times 86,400 \times 2 \times 10^{-3}$ = 172.8	365	0.71	44,781	$3.0 \times 86,400 \times 2 \times 10^{-3}$ = 518.4	365	0.71	134,343	
E	E_1	$2.0 \times 86,400 \times 2 \times 10^{-3}$ = 345.6	365	0.71	89,562	$2.0 \times 86,400 \times 2 \times 10^{-3}$ = 345.6	365	0.71	89,562	
	E_2	$1.0 \times 86,400 \times 2 \times 10^{-3}$ = 172.8	365	0.71	44,781	$3.0 \times 86,400 \times 2 \times 10^{-3}$ = 518.4	365	0.71	134,343	

* NOTE : 1) Chemicals cost is estimated for liquid chlorine for disinfection.
2) Average dosage rate is assumed to be 2 ppm.
3) Sewage quantity includes total amount of San Juan STP.

7.6.3 Technical Evaluation

(1) Alternative A (Pumping and Gravity Flow)

The disadvantages of this alternative are presented as follows:

Pump capacity required at Point B is Q=2.6 ~ 1.1 m^3/s with H=59m. This requirement is categorized under a specification of special pumps for wastewater use with a high pump head. Maintenance cost will, therefore, increase because of the necessity of importing spare parts. Moreover, power cost will be US$ 460 ~ 400 thousand, which additional financial burden SEDAPAL have to shoulder.

On the other hand under Plan B treated sewage is pumped from elevation H=+50m to H=+100m. However, at elevation H=+50m where sewage transmitted by gravity is treated, half of the treated volume of sewage is reused for irrigation. Therefore, capacity necessary to pump treated sewage up to H=+100m is half that of Plan A. Correspondingly, the required cost for power consumption will also be half.

It may be concluded that Plan A is inferior to alternative B from the view point of economy and finance due to higher construction and maintenance cost for pumps.

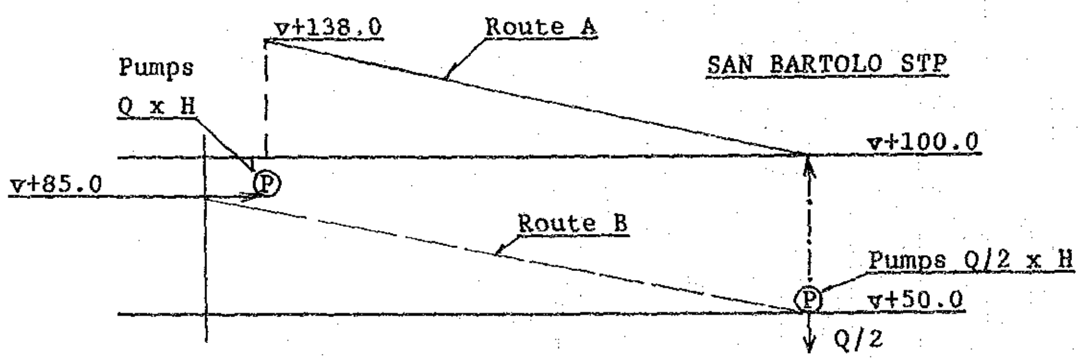


FIGURE 7-25 COMPARISON OF PUMP FACILITY IN PLAN A AND PLAN B

Conclusion:

This plan is not recommendable for Phase I because of high O & M (Power cost and repair cost) and replacement cost.

(2) Alternative B (Gravity Flow)

This plan seems to be an optimum plan because the sewage can be transmitted by gravity flow without pumps. However, disadvantages of this plan are as follows:

a) Possibility of passage through the site of the Pachacamac Ruins has not been confirmed by the time of detailed investigation of Ruins.

b) Possibility of passage through the site of the Explosives Plant (Explosivos S.A.) in San Bartolo has not been confirmed. According to SEDAPAL, there are possibilities of getting approval after the discussion with the superior authority.

c) Effect of accident

The planned inverted siphon will traverse an 8 km stretch of the Panamerican Highway. In case an accident occurs along that stretch, damage on the transportation system will be serious.

Conclusion:

It is hard to adopt this plan for Phase I because of the dim possibility of getting approval for passage through the Pachacamac ruins.

However, if a detailed investigation in the future, reveals the possibility of passing through the ruins, this may prove to be an optimum plan in accordance with a selected pipe material.

(3) Alternative C (Gravity Flow + Pumping)

In this plan, the surrounding area will be developed first and then the San Bartolo Area will be developed in the future. This phased development plan is a distinct feature of alternative C in contrast with alternatives A and B which give priority to development of San Bartolo.

The advantages of this plan are as follows:

a) It will contribute to the accelerated installation of sewerage system and improvement of environmental conditions in the area from San Juan to Villa El Salvador.

b) It will enable the supply of 0.5 m^3/s of treated sewage to Villa El Salvador area where raw sewage is utilized for irrigation at present.

c) Even though a small amount of fund is invested, remarkable benefits are expected.

On the other hand, disadvantages of this plan are as follows:

a) Same as in the case of alternative A, due to usage of pumps.

b) Same as in item c) of alternative B regarding inverted siphon.

c) It is hard to carry out improvement work on the San Juan STP from an emotional point of view because the STP has been in operation for about 20 years and it has become a symbol of environmental improvement projects in Peru.

Conclusion:

Plans C_1 and C_2 are inadequate for phase I because a pumping facility is necessary and it gives the same disadvantages as alternative A has.

Plan C_3 is not recommendable for phase I because of the need for passage through the Pachacamac Ruins.

In plan C_3', however, transmission of the treated sewage to the destination by gravity without a pumping facility is possible because of the high potential of proposed STP at the site of c).

The alternative plan with a proposed STP at c) is also optimum even though the project area is limited to the Villa El Salvador.

(4) Alternative D (Gravity Flow + Pumping)

Project scale and content during Phase I is almost the same as that of plans C_1 and C_2. Therefore, advantages and disadvantages of Plan D are almost the same as that of those plans because of the need for passage through the Pachacamac Ruins and necessity of pumping facilities.

(5) Alternative E (Gravity Flow)

The sewage can be transmitted to the destination by gravity flow and inverted siphon without pumps.

In Phase I, 2.0 m^3/s of sewage (plan E_1) will be transmitted through the same route as that of alternative A. Adoption of inverted siphon cannot be avoided due to topographic constraints. However, in case of accident, damage will be less than that which may be incurred under alternative A.

(6) Conclusion of Technical Evaluation

Comparison of technical evaluation of each alternative is shown in TABLE 7-23. Based on this Table, alternative E is judged to be superior.

TABLE 7-23 COMPARISON OF TECHNICAL EVALUATION

PLAN		PHASE I PLANNED SEWAGE QUANTITY (m^3/s)	TECHNICAL EVALUATION	REMARKS
A	A_1	4.0	*	Devaluation due to necessity of Pumping Facility.
	A_2	2.0	*	
	A_3	1.0	*	
B	B_1	4.0	*	Devaluation due to dim possibility of approroval for passing of transmission line.
	B_2	2.0	*	
	B_3	1.0	*	
C	C_1	2.33	*	Same as Alternative A and B.
	C_2	1.83	*	- ditto -
	C_3	0.83	*	- ditto -
	C_3'	1.0	*	Devaluation due to long siphon.
D	D_1	1.5	*	Devaluation due to necessity of Pumping Facility.
	D_2	1.0	*	
E	E_1	2.0	***	
	E_2	1.0	***	

Legend : *** Superior ** Intermediate * Inferior

Considerations on Inverted Siphon Structure:

To convey the sewage generated in southern part of Lima to San Bartolo along the shortest way, 9.8 km long inverted siphon crossing the Rio Lurin shall be constructed. However, both in Peru and Japan, there is a few cases of long-span inverted siphon for transmission of sewage. Although both countries have experiences of the installation of short-span inverted siphon, there is no case with such a long inverted siphon for sewage works. Thus, following matters shall be considered deliberately prior to implementation of the Project to secure the safety from technical viewpoint:

a) Study on another route which does not need inverted siphon

In this Project, transmission line of recommended plan runs outside of explosives plant site (75 m far away from foundry of the site due to a restriction of the explosives plant). If the passing through the site become possible, safer gravity flow plan will be possible.

b) Pipe materials for inverted siphon

Study on optimum pipe materials for inverted siphon against corrosion and leakage, and on joint method against leakage and joint separation, and the like, shall be conducted.

c) Maintenance of inverted siphon

Study on effective maintenance and reliability in the long run operation of grit chamber at inlet of inverted siphon and of blow-off facility shall be conducted, for instance, on measures for blowing-off of sewage, and on possibility of removal of grit and deposit in the sewer.

d) Putrefaction of sewage during passage through the inverted siphon

Putrefaction of sewage is expected during the long period flowing down under the anaerobic condition. Thus investigations and study on following matters shall be conducted:

- Influence to structure by hydrogen sulfide gas generating from sewage.
- Influence to treatment efficiency by putrefaction of organic matter.

7.6.4 Selection of Optimum Plans

Construction cost, O&M cost and technical evaluation of each alternative are summarized in TABLE 7-24.

TABLE 7-24 COMPARISON OF EVALUATION

PLAN		PHASE I PLANNED SEWAGE Q'TY (M^3/S)	CONSTRUCTION COST	O&M COST	TECHNICAL EVALUATION	COMPREHENSIVE EVALUATION
A	A1	4.0	***	*	*	
	A2	2.0	*	*	*	
	A3	1.0	**	*	*	
B	B1	4.0	***	***	*	
	B2	2.0	**	**	*	
	B3	1.0	***	***	*	
C	C1	2.33	*	*	*	
	C2	1.83	*	*	*	
	C3	0.83	*	*	*	
	C3'	1.0	*	*	*	
D	D1	1.5	*	*	*	
	D2	1.0	*	*	*	
E	E1	2.0	**	**	***	***
	E2	1.0	**	**	***	**

Legend : *** Superior ** Intermediate * Inferior

The following are concluded from TABLE 7-24:

(1) The alternatives with pumping facilities are not recommendable because O & M costs (power + repair) and replacement cost of pumps (25 to 30 years) are expensive.

(2) The alternatives in which the transmission line will pass through the Pachacamac Ruins is not recommendable for Phase I. However, there may be possibilities of passing through the ruins after detailed investigations. Therefore, based on the discussions in Subsection 7.6.4, plan E_1 is selected as an optimum plan for further study.

The planned route and longitudinal section of plan E_1 are presented in attached FIGUREs 7-26 to 7-28 as a recommended plan.

CHAPTER 8

POLLUTION ANALYSIS FOR THE COAST OF CHIRA

CHAPTER 8 POLLUTION ANALYSIS FOR THE COAST OF CHIRA

8.1 Introduction

Sea water pollution along the coastal areas of Metropolitan Lima had become severe by 1980. In 1984, the Direction Tecnica de Salud Ambienta (Environmental Health Technical Dept., hereinafter referred to as DITESA), Ministerio de Salud (Ministry of Health), started a survey and research project on these pollution problems.

The said investigation was carried out to determine measures to be taken against pollution in order to preserve the environment of coastal areas to ensure their safety for public use. Activities of the project staff consisted of a biological and physical survey and research covering the entire stretch of the Metropolitan Lima coastal area. An attempt was later on made to estimate the present and predict the future pollution levels using a newly developed computer-aided simulation model.

In this particular Study, biological, chemical and physical survey and research were undertaken of the sea area along the coast affected by sewage discharge from the Colector Surco.

Simulation model analysis was conducted also with the use of the above-mentioned model developed by DITESA in 1984, based on the results of measurements done in this Study, for prediction of future pollution levels in areas utilized for swimming. Actual computation for the simulation was performed by DITESA.

8.2 Present Sea Water Pollution Condition

8.2.1 Bacteria

In order to understand the present condition of sea water quality, analyses especially on bacteriological items, like Total Coliform, Fecal Coliform and Salmonella were conducted by SEDAPAL several times during the Study period at designated sampling stations shown in FIGURE 8-1. Results of analyses are summarized in TABLE 8-1 (refer to APPENDIX 22 for detail).

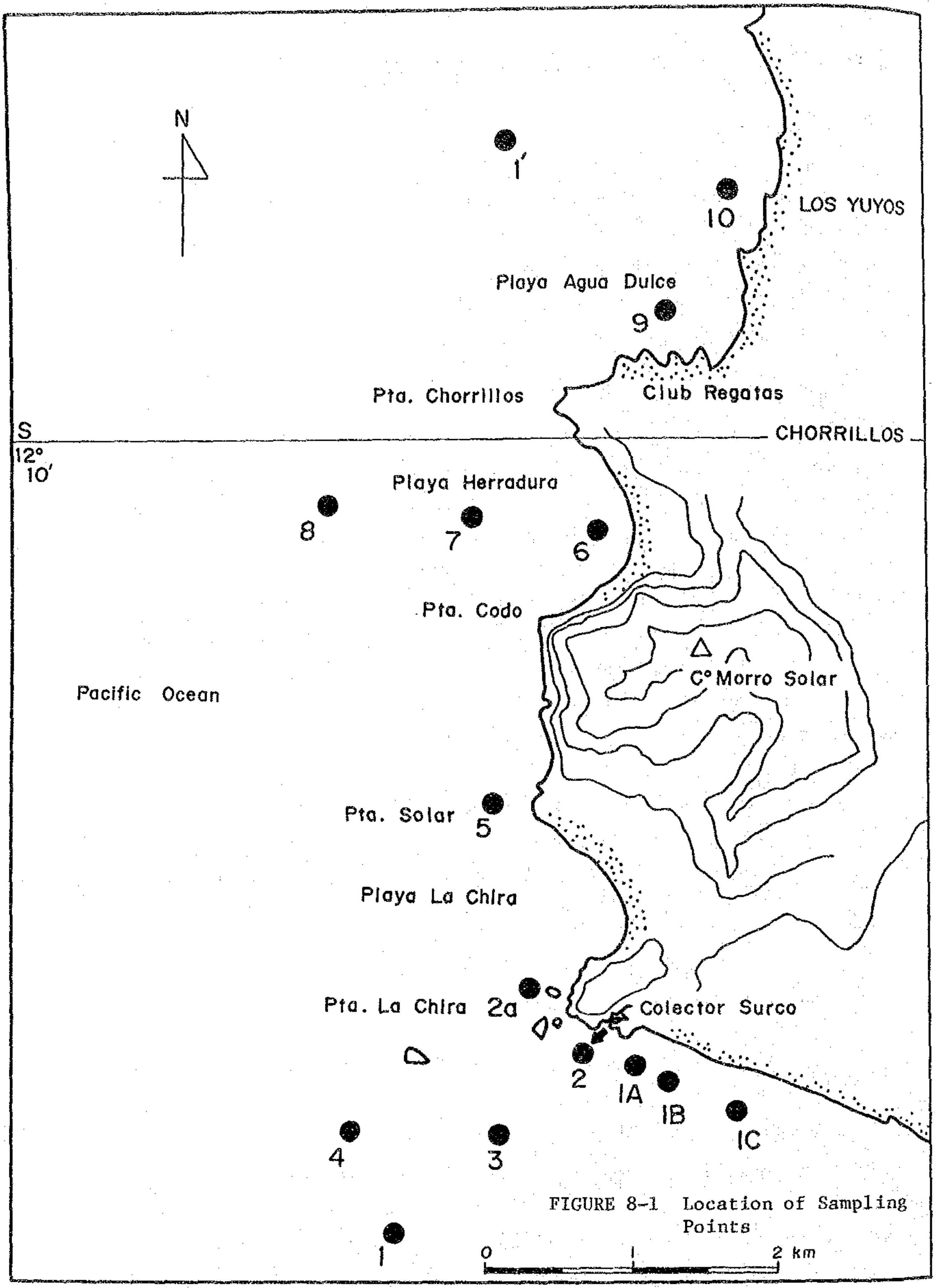


FIGURE 8-1 Location of Sampling Points

TABLE 8－1　Quality of Sea Water

Analysed Items, Date(1989)	Sampling Point No.															
	1	1'	1A	1B	1C	2-0m	2-3m	2-5m	2a	3	4	5	6	7	8	9	10
Total Coliform																	
5／15	—	—	—	—	—	7.5×10^{7}	—	—	1.5×10^{3}	—	—	2.1×10^{4}	< 1	—	—	2.1×10^{2}	—
5／23	—	—	—	—	—	1.1×10^{7}	1.5×10^{5}	$<2.4\times10^{3}$	—	—	—	7×10	2.3×10	—	—	—	9
5／29	—	4.3×10	—	—	—	2.4×10^{7}	—	—	—	2.4×10^{7}	$<2.4\times10^{3}$	2.4×10^{4}	2.3×10	$>2.4\times10^{3}$	$>2.4\times10^{3}$	9	15
6／5	3.9×10	—	—	—	—	2.4×10^{7}	—	—	—	2.4×10^{5}	4	2.4×10^{4}	2.4×10^{2}	4×10	2.4×10^{2}	2.4×10^{3}	4.3×10
6／12	—	—	—	—	—	1.1×10^{7}	—	—	—	9.3×10^{5}	1.5×10	2.4×10^{3}	2.3×10	4×10	7	—	2.3×10^{2}
10／16	—	—	—	—	—	2.3×10^{6}	—	—	—	1.1×10^{6}	1.1×10^{4}	$>2.4\times10^{4}$	4.6×10^{2}	4.6×10^{2}	4.3×10	9	2.4×10
10／23	—	—	—	—	—	9.3×10^{6}	—	—	—	4×10^{4}	4×10^{2}	2.3×10^{4}	1.1×10^{3}	1.1×10^{3}	1.1×10^{3}	2.4×10^{2}	2.3×10
10／30	—	—	4.6×10^{4}	4	< 1	4.3×10^{6}	—	—	—	4×10^{2}	—	9.3×10^{3}	4.6×10^{2}	7.5×10	1.1×10^{4}	—	—
Fecal Coliform																	
5／23	—	—	—	—	—	4.6×10^{6}	1.2×10^{5}	$<2.4\times10^{2}$	—	—	—	7×10	2.3×10	—	—	—	9
5／29	—	2.3×10	—	—	—	1.1×10^{7}	—	—	—	4.6×10^{5}	$<2.4\times10^{3}$	2.4×10^{4}	< 1	$>2.4\times10^{3}$	$>2.4\times10^{3}$	4	9
6／5	2.3×10	—	—	—	—	2.4×10^{6}	—	—	—	2.4×10^{5}	4	4.6×10^{3}	< 1	4×10	2.4×10^{2}	2.4×10^{2}	9
6／12	—	—	—	—	—	2.4×10^{6}	—	—	—	2.4×10^{4}	4	2.4×10^{3}	< 1	4×10	4	—	2.3×10
10／16	—	—	—	—	—	2.3×10^{6}	—	—	—	1.5×10^{5}	1.1×10^{4}	1.1×10^{4}	2.4×10^{2}	4.6×10^{2}	4.3×10	< 1	4
10／23	—	—	—	—	—	4.3×10^{6}	—	—	—	4×10^{4}	4×10^{2}	2.3×10^{4}	4.6×10^{2}	1.1×10^{3}	1.1×10^{3}	4.3×10	< 1
10／30	—	—	4.6×10^{4}	< 1	< 1	9×10^{5}	—	—	—	4×10^{2}	—	9.3×10^{3}	4.6×10^{2}	2.3×10	9.3×10^{2}	—	—
Salmonella Bacteria																	
6／12	—	—	—	—	—	2.4×10^{2}	—	—	—	< 1	—	< 1	—	—	—	—	—
10／16	—	—	—	—	—	9.3	—	—	—	1	< 1	—	—	—	—	—	—
10／30	—	—	2.1	< 1	< 1	—	—	—	—	—	—	—	—	—	—	—	—

The concentration of coliform bacteria in the sea water varies widely with the location of sampling point and date of sampling. From the results of the analyses, it can be inferred that the sea water quality at Point Nos. 1A, 2, and 3 were seriously affected by the raw sewage discharged from the Surco Outfall, the Fecal Coliform level being measured at 10^7 to 10^5 MPN/100ml, which far exceeds the water quality standard. Point Nos. 4, 5, 7 and 8 also showed a rather high Fecal Coliform concentration of 10^4 to 10^2 MPN/100ml. On the other hand, at Point Nos. 2 and 9 the high Fecal Coliform concentration occurred occasionally. Traces of raw sewage discharge are very minimal at Point Nos. 1, 1', 1B, 1C and 10. Salmonella bacteria were also found at Point Nos. 1A, 2 and 3, where many commercial fishing boats gather and catch fish.

Although these analyses were conducted only during the winter season, it is inferred from the findings and observations that the raw sewage discharged from the Surco Outfall flows westward for some distance offshore then changes direction with the north treading sea current. It may therefore be, inferred that the coastal area south of the Surco Outfall is not affected by pollution caused by its raw sewage discharge.

In addition to the analyses by the Study team, CEPIS conducted a series of sea water quality analysis on Total and Fecal Coliform for 27 weeks from October 1986 to April 1987 (refer to APPENDIX 22). In the survey, sea water sampling was carried out once a week at 22 sites along the coast of Lima extending from south to north along the Pacific Ocean. Based on the results of that survey, the fecal coliform concentrations measured at sites near the sampling points of the JICA Study often exceed the desirable water quality standard. The survey was conducted in summer, during which period many people swim and surf in the vicinity of these points. Thus, the survey implies that measure against water pollution along these areas must be taken as soon as possible.

8.2.2 Heavy Metals

Concentrations of heavy metals, like Hg, Cu and Cd, in shell fish collected at Agua Dulce beach has been analyzed from 1985 to 1988 by engineers of UNEP in cooperation with CEPIS (refer to FIGURE 8-2).

Based on the results of this survey, mercury (Hg) and copper (Cu) levels were very much lower than the permissible limits of 500 μg/kg and 100 mg/kg, respectively.

Cadmium (Cd) content, however, was much higher than the limit of 0.05 mg/kg, and it is possible that the sewage from the Colector Surco including industrial wastewater influenced this high concentration.

On the other hand, copper and cadmium concentrations in the sea water around Callao Port were measured from 1974 to 1986 by IMARPE. Resulting values were 9.7 - 26.7 μg/l for copper and 1.58 - 51 μg/l for cadmium.

From the above data, it is apparent that heavy metals are observed in great concentrations by organisms from the surrounding water, although the sampling dates and locations were not exactly same.

It is necessary, therefore, that heavy metals originating from sewage including industrial wastewater be pressed down as low as possible. Decrease to some extent of heavy metal concentrations in sea organisms is expected even with a small reduction of the volume of sewage discharge from Colecter Surco.

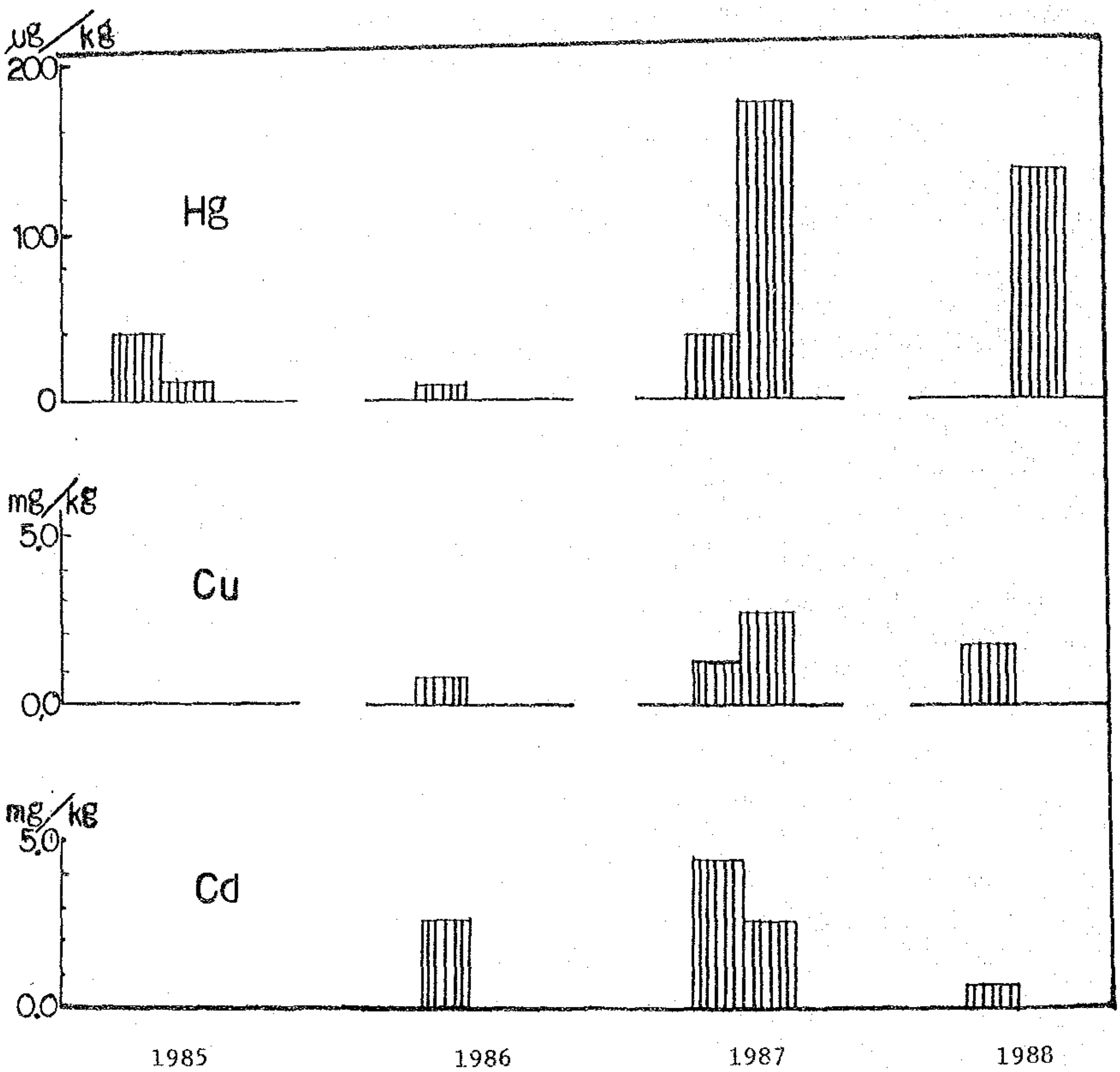


FIGURE 8-2 Hg, Cu, and Cd in Shell Fish

8.3 Computer Simulation

8.3.1 Adopted Parameter

Fecal coliform bacteria was used as a parameter in the simulation model, because it is one of the major parameters of water quality standard in Peru, and is judged to be directly related to the water pollution caused by sewage.

8.3.2 Outline of the Simulation Model

Box Mixing Model developed by DITESA in 1984 was used for this simulation. Outline of this simulation model is as follows:

(1) Assumptions

a) Parameter does not vary with time; only spatial distribution varies.

b) Since there were no vertical differences in temperature (no thermocline), the bacteria are assumed to be distributed only in the X and Y directions (horizontal).

c) The survey area was divided into 77 segments, with smaller segments set near the Surco Outfall.

d) Balance formula of the flowing seawater volume

1) Volume of flowing seawater is balanced.
2) At the segment where the sewage from Colector Surco flows into, the total volume is calculated as the sum of the sewage volume and the volume of flowing seawater.

e) Balance of coliform number in the segments

1) $Q_i \times C_i$: Balance formula

Q_i: Volume of flowing seawater between the segments

C_i: Coliform number in each segment

2) Boundary condition of each segment

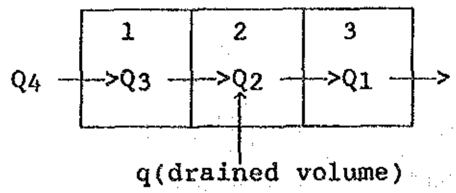
(i) (Discharge volume of sewage) x (Fecal coliform number) is used for the coliform value from Colector Surco.

(ii) In segments 73, 75, 76 and 77, which are located on the part of the survey area from where the seawater flow comes, coliform numbers are calculated.

(2) Balance Formula

a) Balance of the flowing seawater volume

Flowing water volume : Q_1, Q_2, Q_3

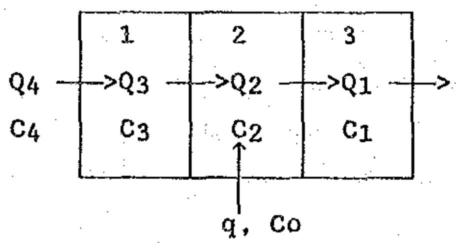


$$1: Q_4 = Q_3 \; \ldots\ldots \; 1)$$
$$2: Q_3 + q = Q_2 \; \ldots \; 2)$$
$$3: Q_2 = Q_1 \; \ldots\ldots\ldots \; 3)$$

q : drained volume of sewage

b) Balance of substances

Balance of the flowing seawater volume x coliform concentration



$$1: Q_4 C_4 = Q_3 C_3 \; \ldots\ldots\ldots\ldots \; 4)$$
$$2: Q_3 C_3 + q\, C_o = Q_2 C_2 \; \ldots \; 5)$$
$$3: Q_2 C_2 = Q_1 C_1 \; \ldots\ldots\ldots\ldots \; 6)$$

If the values of Q_4, C_4, q and Co are known, the values of C_1 to C_3 and Q_1 to Q_3 are decided from equations 1) to 6).

If decreasing rate of C is known as K (mortality rate), then:

$$4) \text{ is } Q_4 C_4 = Q_3 C_3 + K.C_3 \text{ .... } 4')$$
$$5) \text{ is } Q_3 C_3 + q.Co = Q_2 C_2 + K.C_2 \text{ ... } 5')$$
$$6) \text{ is } Q_2 C_2 = Q_1 C_1 + K.C_1 \text{ .... } 6')$$

Velocity of seawater in the survey area is set this time at 0.125 knot, which was used in the analysis in 1984 by DITESA based on actual measurement.

FIGURE 8-3 shows the balance of flowing seawater volume in each segment calculated by DITESA using the measured values in the area.

8.4 Computation Results

8.4.1 Data for Simulation

(1) Coliform

Coliform numbers in the sewage discharging from the Surco Outfall are measured at three main sewers, namely Colector Surco, Colector Circunvalacion, and Colector Balnearios del Sur. Measurements are done three times generally at a frequency of every 15 minutes for 24 hours (refer to APPENDIX 5). Sewage flow measurement was performed simultaneously as described in section 5.2.

Since the fecal coliform numbers at the three spots were different and varied with time, the average coliform concentration was computed with reference to the above-mentioned variation in coliform concentration and the sewage flow of each main sewer. Analyses results are summarized in TABLEs 8-2 to 8-4.

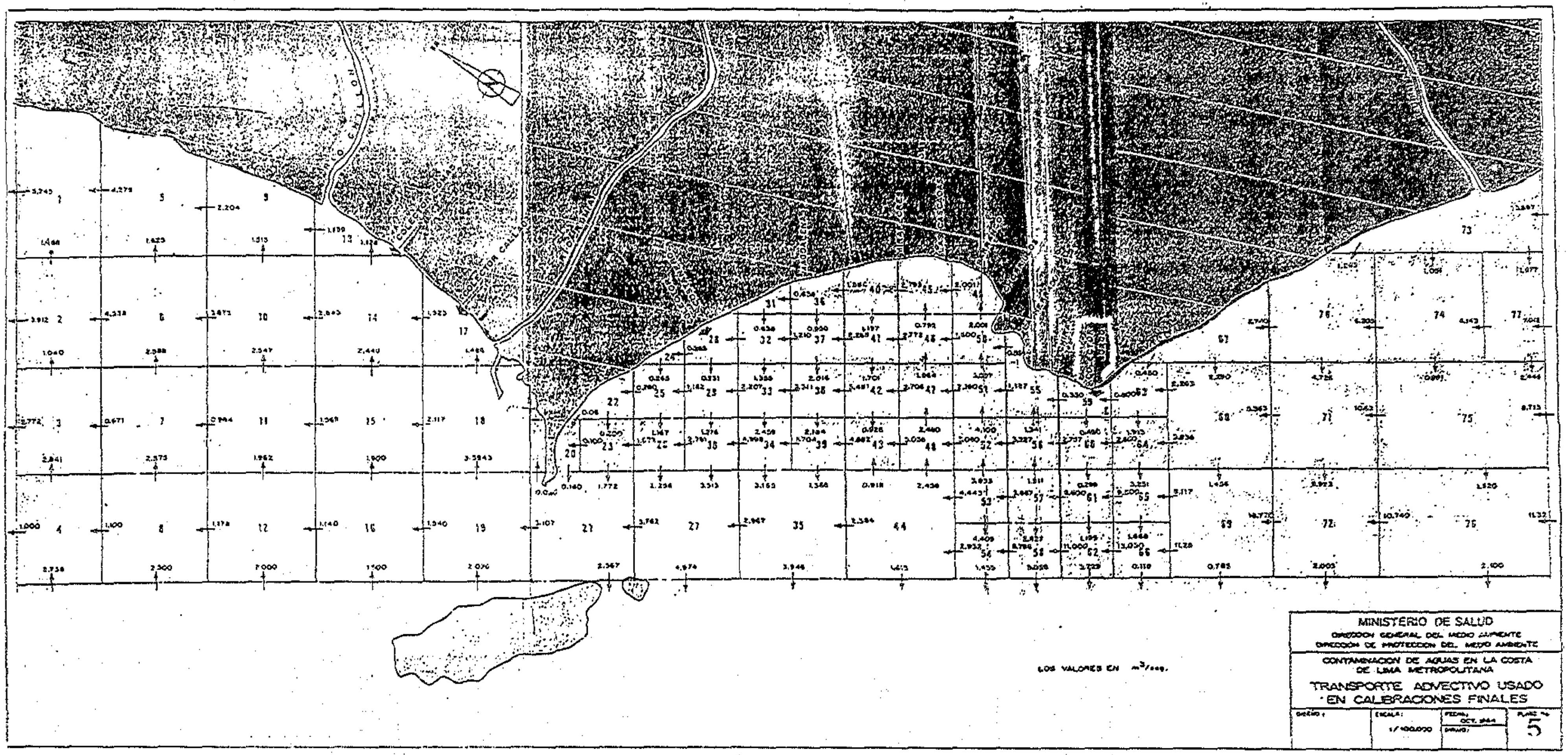


FIGURE 8 - 3 BALANCE OF SEAWATER FLOW BY SEGMENTS

TABLE 8-2 Coliform Number and Sewage Flow (May 31 - June 1)

		Main Sewer	Surco	Circun.	B. Sur	Total
Sewage Flow (m^3/sec)	Maximum	4.929	1.454	0.305	6.569
	Minimum	2.769	0.839	0.082	3.756
	Average	4.058	1.134	0.178	5.370
Fecal Coliform (MPN/100ml)	Maximum	$4.6x10^7$	$4.6x10^7$	$2.4x10^7$	$4.5x10^7$
	Minimum	$2.4x10^7$	$2.4x10^7$	$2.4x10^7$	$2.4x10^7$
	Average	$3.7x10^7$	$3.4x10^7$	$2.4x10^7$	$3.6x10^7$

TABLE 8-3 Coliform Number and Sewage Flow (June 19 - June 20)

	Main Sewer	Surco	Circun.	B. Sur	Total
Sewage Flow* (m^3/sec)	Maximum	4.929	1.454	0.305	6.569
	Minimum	2.769	0.839	0.082	3.756
	Average	4.058	1.134	0.178	5.370
Fecal Coliform (MPN/100ml)	Maximum	$1.1x10^8$	$4.6x10^7$	$1.1x10^8$	$9.7x10^7$
	Minimum	$4.3x10^6$	$9.3x10^6$	$2.4x10^7$	$6.3x10^6$
	Average	$4.5x10^7$	$3.9x10^7$	$5.1x10^7$	$4.4x10^7$

*: Sewage flow measured on May 31 to June 1 was applied.

TABLE 8-4 Coliform Number and Sewage Flow (Oct. 19 - Oct. 20)

	Main Sewer	Surco	Circun.	B. Sur	Total
Sewage Flow (m^3/sec)	Maximum	4.477	1.612	0.296	6.324
	Minimum	2.313	0.841	0.076	3.240
	Average	3.625	1.157	0.181	4.963
Fecal Coliform (MPN/100ml)	Maximum	$2.4x10^8$	$2.4x10^8$	$4.3x10^7$	$1.9x10^8$
	Minimum	$9.0x10^6$	$4.0x10^6$	$4.0x10^6$	$7.5x10^6$
	Average	$7.5x10^7$	$9.6x10^7$	$3.2x10^7$	$7.8x10^7$

Based on the above tables, the concentration of fecal coliform in the raw sewage varies between the levels of 10^6 to 10^8 MPN/100ml. One of the reasons for the high value in October is surmised to be due to the stoppage of clear water intrusion coming from the irrigation canal. For the simulation, fecal coliform concentration was decided as the average value of three measurements as follows:

$$\text{Average fecal coliform concentration} = (3.6+4.4+7.8) \times 10^7 \div 3$$
$$= \text{approx. } 5 \times 10^7 \text{ MPN/100ml}$$

(2) Sewage Flow

The average sewage flow discharged from the Surco Outfall was set at 5.0 m^3/sec for the simulation based on discussions in section 5.2.

8.4.2 Simulation Results

FIGURE 8-4 shows the simulation results as contour lines of 1,000 MPN/100ml of fecal coliform concentration for various volumes of sewage ranging from 2.5 to 6.5 m^3/s with an interval of 0.5 m^3/s.

Fecal coliforms of 5.0 x 10^7 MPN/100ml and velocity of 0.125 knot are used for these simulations.

Based on the computation results, contamination caused by the raw sewage discharge from Surco Outfall spreads northward, and the contour line which can be perceived as the boundary of area affected by pollution reaches up to the Club Regatas at present discharge condition.

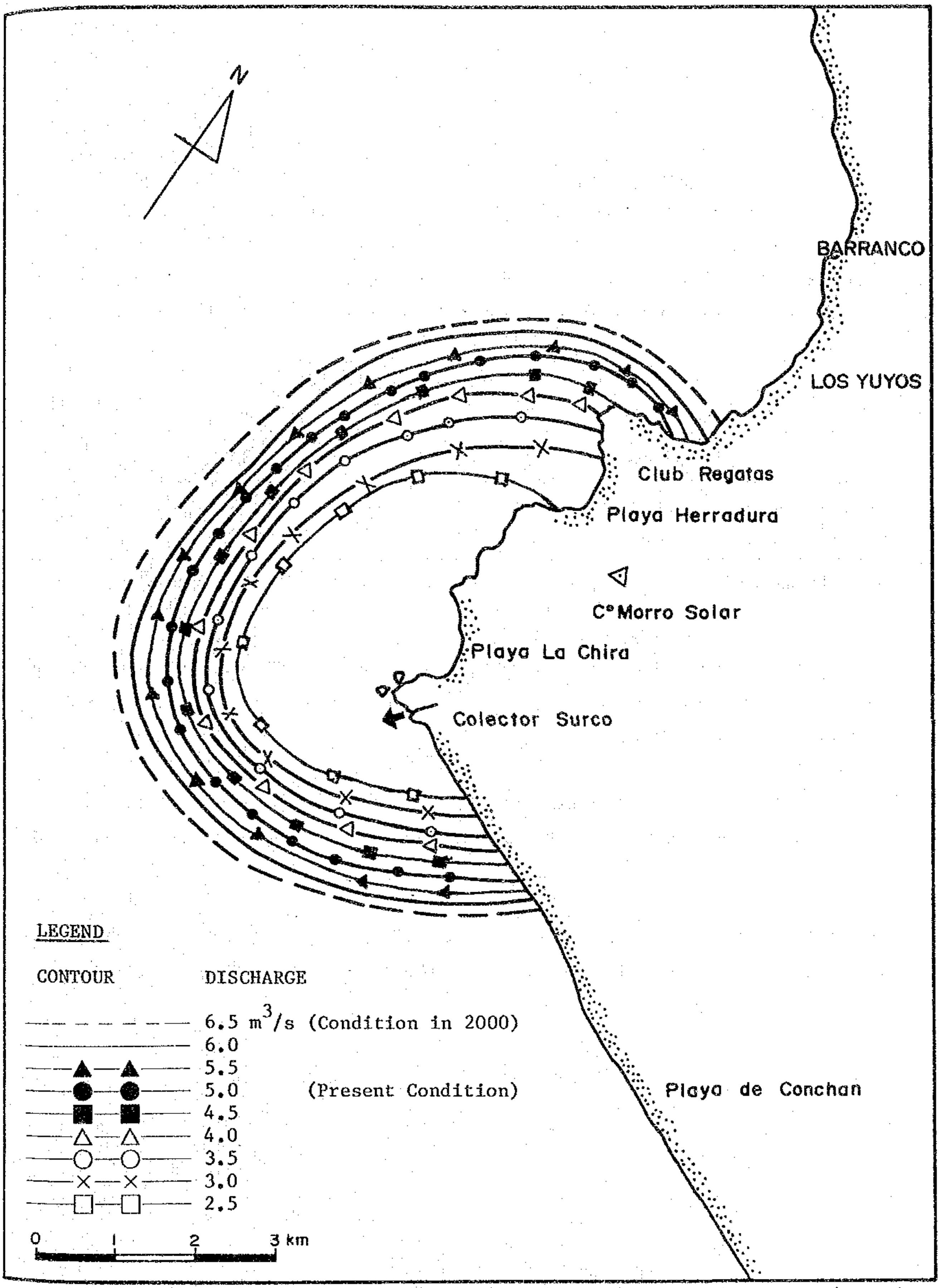


FIGURE 8-4 SIMULATED CONTOUR LINE OF FECAL COLIFORMS (1,000 MPN/100ml)