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APPENDICES

APPENDIX-A

PRELIMINARY CALCULATION OF THE YIELD OF THE DAM PROPOSED IN THE CHOQUEYAPU RIVER BASIN

1. PURPOSE OF THE CALCULATION

At the final stage of the first site study in La Paz, the Study Team was advised by HAM/SAMAPA of an idea to construct a water supply dam in the Choqueyapu river basin. Also they suggested it would be possible to operate the dam so as to increase the flowrate of the Choqueyapu River in the dry season.

If such a dam and its operation to supply dilution water is in fact feasible, the expected increase of the flowrate should be reflected in the pollution analysis model in the Study. The calculations of this Appendix were performed to obtain a rough idea on the possible increase of the flowrate by the proposed dam and its effect on pollutant concentrations.

2. BASIC ASSUMPTIONS OF THE CALCULATIONS

(1) Dam

No technical information on the proposed dam has been provided to the Study Team. Therefore, several assumptions were necessary for the calculation:

- Volume: 10,000,000 m^3
- Average depth: 10 m
- Catchment Area: 80 % of the catchment area of the Achachicala flow observation point, supposing that the dam will be located upstream of the observation point.

(2) Inflow to Dam

This is assumed from the observed monthly mean flow rates at the Achachicala station (for data source, refer to Table 2.1.11 of the Progress Report, June, 1992).

from which,

Monthly inflow (m^3/month) = Flowrate at Achachicala (m^3/sec) * 80% * 86400 * 30

(3) Evaporation Losses

Calculated from the observed evaporation rate in Central La Paz (Ref. Table 2.1.8 of the Progress Report, June, 1992).

from which,

Monthly Evaporation Loss (m^3/month) =
Evaporation rate (mm/day) * 30 / 1000 * 10,000,000 / 10

(4) Discharge Rate

Discharge rates from the dam are assumed to vary from 0.5 m^3/sec to 1.0 m^3/sec.

3. WATER BALANCE

The monthly water balance of the dam was calculated by the following equation:

$$V_n = V_{n-1} + Q_{in} - Q_{out} - Q_{evp}$$

where,

V_n: Storage volume at the end of the month. (m^3)

V_{n-1}: Storage volume at the end of the previous month (m^3)

Q_{in}: Monthly inflow (m^3)

Q_{out}: Monthly Discharge (m^3)

Q_{evp}: Monthly evaporation loss (m^3)

4. RESULTS AND DISCUSSION

Results are shown in the Table and Figure on the following page. The results show that the storage volume would be negative after June at the discharge rates more than 0.8 m^3/sec and reaches almost zero at a discharge rate of 0.7 m^3/sec. Therefore, it can be said the maximum discharge rate from the dam would be less than 0.7 m^3/sec.

The minimum flowrate at the Achachicala observation station during the dry season is presently about 0.4 m^3/sec as shown in the Table. Thus, the average yield of the dam would be less than 0.3 m^3/sec.

However, it should be noted that after the dam construction there would rarely be a flow in excess of 0.7 m^3/sec during the entire year, even in the rainy season when flood flows usually occur under present conditions. This would adversely affect the river conditions because flood flows in the rainy season now flush out the sludge and garbage accumulated in the river. It should also be noted that these results have been obtained based on the average monthly flow data for a ten year period. This means that the maximum discharge rate could be smaller in 50 % of the years. Consequently, the average yield of the dam should be estimated at much less than the 0.3 m^3/sec that is obtained by the above calculation.

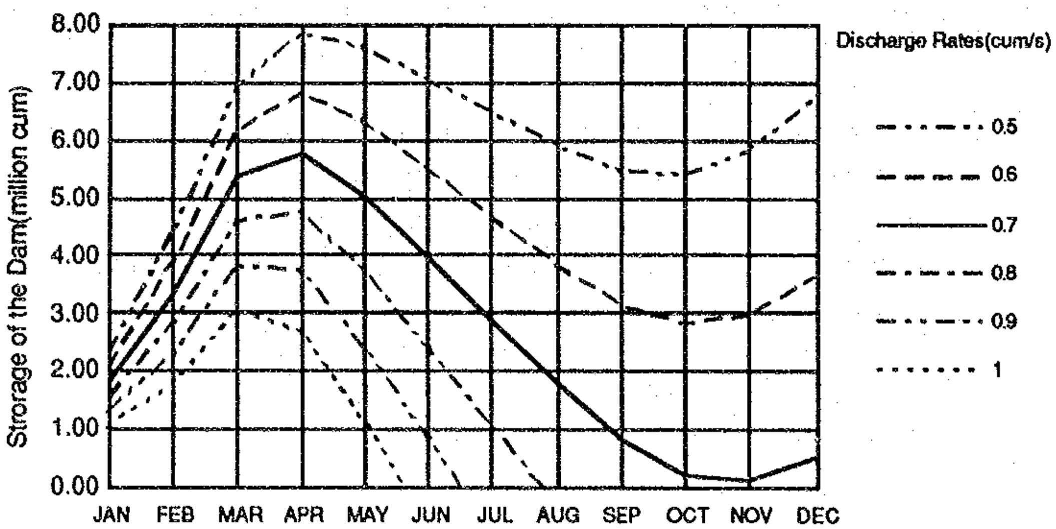
While the available information is not enough to establish the maximum yield, it is apparently not feasible to obtain sufficient dilution water by the proposed dam to come close to achieve the proposed water quality goals. In the absence of definitive planning information for the proposed dam and a positive feasible study, it is not recommended to consider this alternative further at this time.

The effects of dilution water could however be easily included in the pollution analysis model prepared in the further study.

Fluctuation of the Dam Storage to be Constructed in the Choqueyapu River

Month	Flowrate at Achachicala (Mil.m^3/m)	Inflow to dam (80% of Achac.) (Mil.m^3/m)	Loss by Evaporation (Mil.m^3/m)	Net Inflow (Mil.m^3/m)	Discharge Rates (m^3/sec) 0.5	0.6	0.7	0.8	0.9	1.0
JAN	4.69	3.75	0.12	3.63	2.33	2.07	1.82	1.56	1.30	1.04
FEB	4.32	3.46	0.13	3.33	4.37	3.85	3.33	2.81	2.29	1.76
MAR	4.98	3.98	0.13	3.85	6.92	6.14	5.37	4.59	3.81	3.03
APR	2.95	2.36	0.12	2.24	7.87	6.83	5.79	4.76	3.72	2.68
MAY	1.48	1.18	0.13	1.05	7.62	6.32	5.03	3.73	2.44	1.14
JUN	1.09	0.87	0.12	0.75	7.07	5.52	3.96	2.41	0.86	-0.70
JUL	1.06	0.85	0.13	0.72	6.50	4.68	2.87	1.05	-0.76	-2.57
AUG	1.06	0.85	0.13	0.72	5.92	3.85	1.77	-0.30	-2.37	-4.45
SEP	1.22	0.98	0.14	0.84	5.47	3.13	0.80	-1.53	-3.87	-6.20
OCT	1.74	1.39	0.16	1.23	5.40	2.81	0.22	-2.36	-4.97	-7.56
NOV	2.33	1.86	0.16	1.70	5.80	2.95	0.10	-2.75	-5.60	-8.45
DEC	2.95	2.36	0.15	2.21	6.72	3.61	0.50	-2.61	-5.72	-8.83

Note: Shaded area: Storage of the dam, million m^3



FLUCTUATION OF THE DAM STORAGES BY DISCHARGE RATES

APPENDIX-B

CONSIDERATIONS ON DIRECT RIVER PURIFICATION

1. INTRODUCTION

Direct river purification is an idea to improve water quality by applying certain purification methods, which are mainly based on artificial acceleration or enhancement of the self-purification functions of a natural river. Since the Choqueyapu River is presently used as a sewage channel, the direct river purification method might be applied to contribute to water quality improvement. If application of direct river purification to the Choqueyapu River water were to be feasible, a Basic Plan could be prepared that improves a water quality in the study area by combined direct river purification with small scale treatment plants for the South Zone.

To determine the feasibility of direct river purification, preliminary designs of a sedimentation method and a gravel contact purification method have been investigated.

2. SEDIMENTATION METHOD

The sedimentation method considered here is to construct simple sedimentation pond(s) in the vicinity of the river stream in the Central zone. River water would be diverted to the ponds by a weir to be constructed in the river. The idea is to purify and return the treated water to the river, and to transfer sludge to drying beds located downstream. Tentative locations of each component are shown in Figure 1 of this Appendix.

2.1 Water Intake Weir

A water intake weir to be constructed to divert water to the sedimentation ponds should not reduce the flood discharge capacity of the river. A tilting type weir is one of the types which fulfill this requirement. Figure 2 shows a schematic drawing of an inflated rubber type tilting weir. The body of the weir consists of rubber inflated by compressed air, which raises the water level of the river so that water can be diverted to the sedimentation facilities. The inflated rubber body has a valve which is controlled by water level. When water level rises and reaches a fixed level, the valve is opened to exhaust the inside air, resulting in a tilting of the weir. By this means, it is possible to intake all of the river water during dry season without reducing the flood discharge capacity of the river during the rainy season.

2.2 Sedimentation Pond

(1) Site

Now used for parking space for garbage collection vehicles (60m x 110m), refer to Figure 1.

(2) Provisional design

i) Criteria

Water flow to be treated:	105,000 m^3/day (including a river flow of 34,500 m^3/day)
Overflow rate of sedimentation pond:	30 m^3/m^2/day

ii) Required surface of the pond

105,000 m3/day / 30 = 3500 m^2

iii) Dimension and number of ponds

10 m (width), 40 m (length), 3 m (effective depth), 9 ponds

iv) Detention time

(3,600 m^2 x 3 m) / 105,000 x 24 = 2.5 hour

v) Sludge withdrawal

Mechanical type

Layout and longitudinal profile of the weir and the sedimentation pond are shown in Figures. 3 and 4 of this Appendix.

2.3 Sludge treatment

i) Sludge generation rate

Water flow: 105,000m^3/day
SS: 300 mg/l
Removal: 50 %
Sludge Concentration: 2 %
300 x 0.5 x 105,000 / 1,000,000 / 0.02 = 787.5 m^3/day

ii) Sludge transfer

Sludge pipe by gravity flow
Diameter 250 mm, length about 7000 m
3 pressure control tanks

iii) Sludge drying bed

Sludge amount (Q): 787.5 m^3/day
Drying period (T): 15 days
Depth (D): 0.2 m
Required Area (A) = Q x T / D = 787.5 x 15 / 0.2 = 5.9 ha
Available land size: Down stream of La Florida A (1 ha) + B (2.5 ha) = 3.5 ha

2.4 Provisional Cost Estimate

Description	Million yen	
Weir (Weir, river protection, screen, intake)	130	
Sedimentation pond		
Civil works	300	
Sludge collector	400	
Miscellaneous works	100	
Sludge pipe (250 mm, 7000m, pressure control tanks)	300	
Sludge drying bed	60	
Total	1,290	(US$10 million)

2.5 Expected effects

In the provisional design mentioned above, wastewater would be taken from the weir to be constructed below Mercado Camacho and would flow to the sedimentation ponds by gravity. Therefore, the wastewater flowing to the Choqueyapu River below the diversion point could not be treated by this method without pumping. The amount diverted from the proposed point is estimated at 60 % of the entire wastewater from the Central zone.

To treat more wastewater without pumping, the facilities must be located further down stream. However, there is no suitable available land along the stream.

The removal efficiency of the sedimentation method is estimated at a maximum of 50 % of BOD. The effect of the method is calculated as below:

Amount of BOD removed by the method = 1 x 0.6 x 0.5 = 0.3.

This means that the BOD concentration of the river would decrease to about 140 mg/l, assuming the BOD of the river is 200 mg/l.

3. GRAVEL CONTACT PURIFICATION

The gravel contact purification is a method to improve water quality by flow through a tank or basin packed with gravel. In this method, pollutants in the wastewater are settled or absorbed onto the surface of gravel which is covered by a biofilm while passing through the void spaces in the gravel. A portion of the pollutants is decomposed by biological reactions and another portion accumulates in the void spaces as sludge. Since it would not be practical to withdraw sludge from the system frequently, the tank must have a large volume to provide for such sludge accumulation.

A schematic drawing of the gravel contact purification tank to be proposed is shown in Figure 5. In conventional Japanese design, the volume for purification is designed to maintain 1 hour detention time, and the volume for sludge accumulation is designed to have a volume for several years accumulation. However, it would be difficult to install such large scale tanks in the Choqueyapu River in the Central area. Thus, the proposed contact tanks would have only several minutes of detention time and a volume for several days of sludge accumulation. However, many small tanks would be required along the stream as shown in Figure 6. Although the expected removal performance by each tank may be small, total removal performance by repeated tanks is expected to be considerable.

3.1 Design of tanks

(1) Dimension and numbers

The tanks would be installed under the existing river bed so as not to reduce the present flood discharge capacity of the river. To match the river bed profile, the length of the tanks would vary from 5 m to 75 m with an average of 25 m as shown in Figure 6. The width would be fixed at 20 m, considering the normal width of the river and the depth at 5 m, 3 m for purification and 2 m for sludge accumulation.

20 tanks could be installed in the open section of the channel below Mercado Camacho.

(2) Volume for purification and sludge accumulation

Supposing the void ratio of gravel is 35 %,
Effective volume for purification: $20 \times 25 \times 0.35 \times 3 = 525\ m^3$
Effective volume for sludge accumulation: $20 \times 25 \times 0.35 \times 2 = 350\ m^3$

(3) Detention time

For dry season flow (105,000 m^3/day),
535 / 105,000 / 24 = 0.12 hour = 7.2 min. per tank with average length.

For total tanks (20 numbers),
7.2 x 20 = 144 min. = 2.4 hr

(4) Removal ratio

Supposing the removal ratio of BOD and SS by an average tank is 5 %, the total removal is;
$(1 - 0.95^{20}) = 0.64 = 64\ \%$.

3.2 Sludge generation and disposal

(1) Sludge generation and accumulation

Supposing the inlet SS is 300 mg/l, the removal per tank is 5 % and the sludge concentration is 2 %, the sludge generation per tank would be,
300 x 0.05 x 105,000 / 1,000,000 / 0.02 = 78.8 m^3/day.

Since the effective volume for sludge accumulation per tank is 350 m^3 as calculated above, days of sludge accumulation would be,
350 / 78.8 = 4.4 days.

(2) Sludge disposal

In the gravel contact method, since sludge is accumulated in the void spaces in the gravel, it cannot be withdrawn from bottom of the tank as is done in sedimentation tanks. In the design of conventional gravel contact tanks, there is usually a large volume for sludge accumulation, so as to provide sludge storage for several years. However, in the proposed system, it is impossible to provide such a large volume of sludge accumulation, because of the high sludge generation rate and the land availability restrictions.

Therefore, in the proposed system, sludge would be discharged at certain intervals (average 4 to 5 days) by back flushing with air from bottom of the tank. This would require installation of aeration pipes at the bottom of the tanks and blower facilities to supply air.

3.3 Provisional Cost Estimate

Each Gravel Contact Tank (including aeration): 36 million yen,
36 million x 20 = 720 million yen (US$6 million)

3.4 Expected Effect

Since the system would be installed in the open stretch of the channel below the Mercado Camacho, most of the wastewaters from the Central zone would be treated. Supposing the BOD concentration of the river at the outlet of the Central zone is 200 mg/l, a reduction to 80 mg/l (BOD removal, 64 %) would be expected.

However, it should be noted that the proposed system would not remove any amount of pollutant from the wastewater as a whole. The removed sludge would be discharged at a certain interval (4 to 5 days), causing worse water quality conditions than the present during the discharge period.

Therefore, the effect of this system would be to change the present condition, which is always bad, to periods of better conditions and periods of worse conditions.

4. DISCUSSION ON THE FEASIBILITY OF DIRECT RIVER PURIFICATION

4.1 Sedimentation ponds

(1) Shortage of land for sludge drying bed site

The necessary land size for sludge drying beds (5.9 ha) apparently exceeds the available land size (3.5 ha). Thus it is judged that this plan is not feasible from the view point of land availability.

(2) Water quality improvement

The water quality improvement by this method is estimated at 30 % of the BOD load from total Central zone, decreasing the present BOD of 200 mg/l to 140 mg/l. Even if shortage in sludge drying bed sites could be resolved, the water quality improvement is not enough to meet reasonable water quality goals.

4.2 Gravel Contact Purification

There is no way to treat sludge removed from water other than discharge it to the river again. Thus this method would not reduce the pollution load to the river. It may reduce the BOD concentration from 200 mg/l to 80 mg/l, but sludge discharges at a certain intervals would cause worse water quality condition than at present.

4.3 Conclusion

Judging from the limited water quality improvement and the incompleteness of these wastewater treatment methods, in that the sludge is not treated, both methods are considered to be inappropriate as a basic plan or components of thereof.

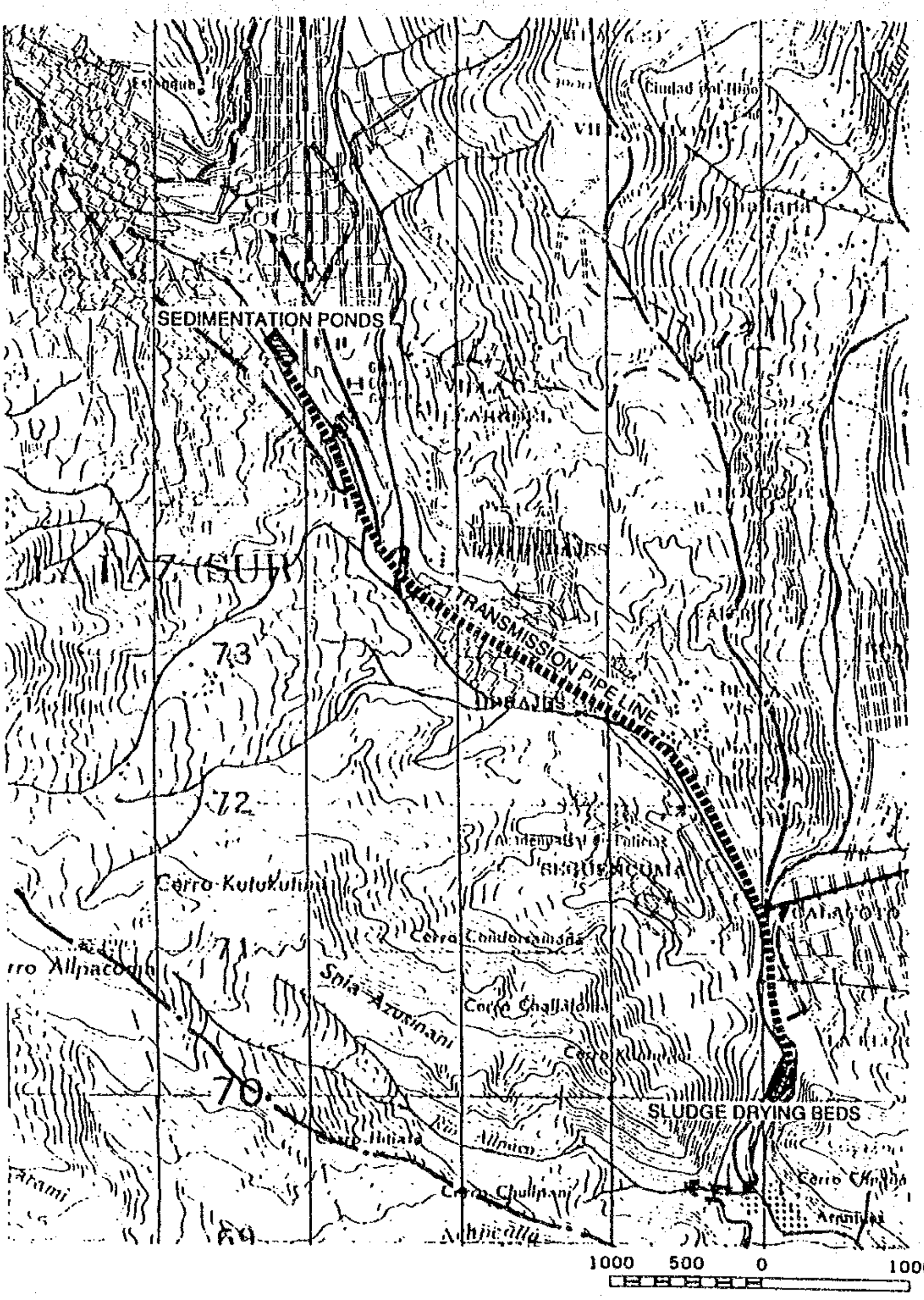


FIGURE 1 LOCATION OF COMPONENTS OF SEDIMENTATION POND METHOD

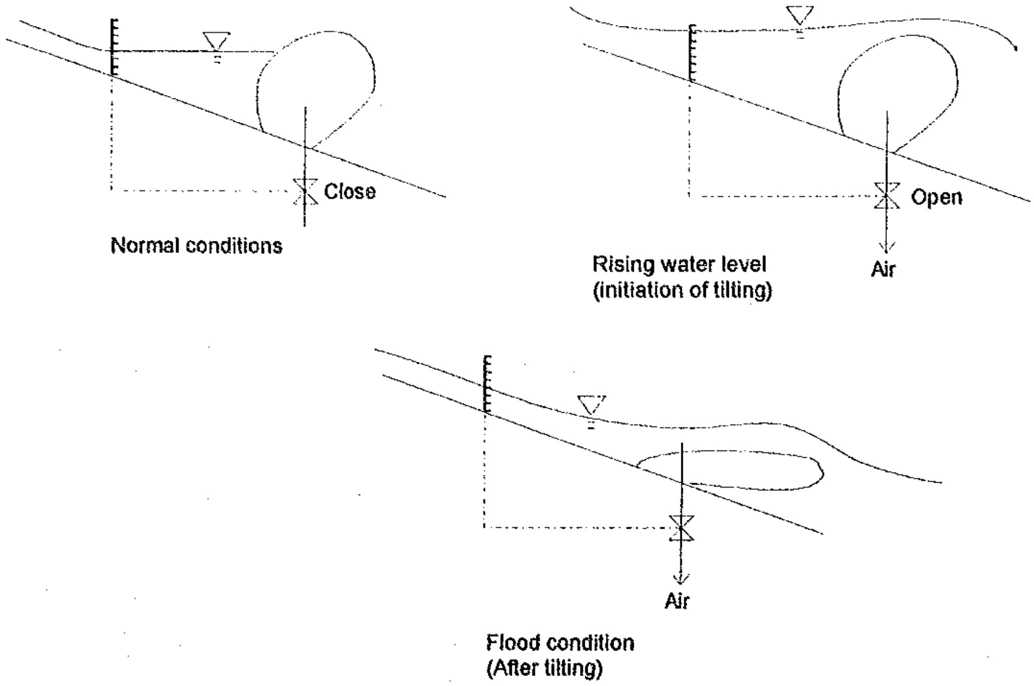


FIGURE 2 FUNCTIONS OF TILTING WEIR (INFLATED RUBBER TYPE)

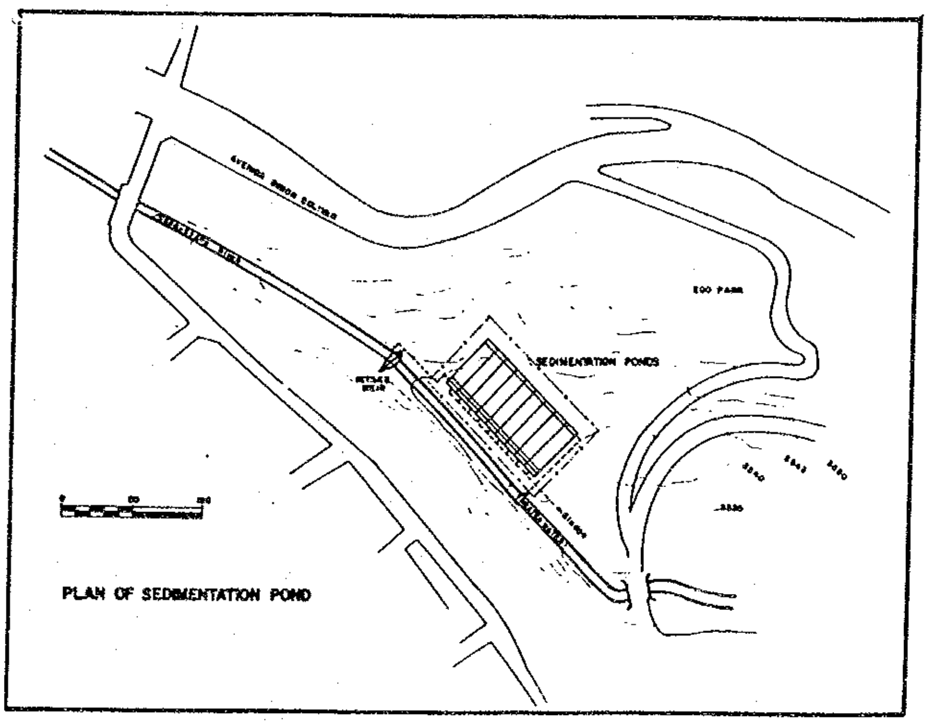


FIGURE 3 PLAN OF SEDIMENTATION PONDS

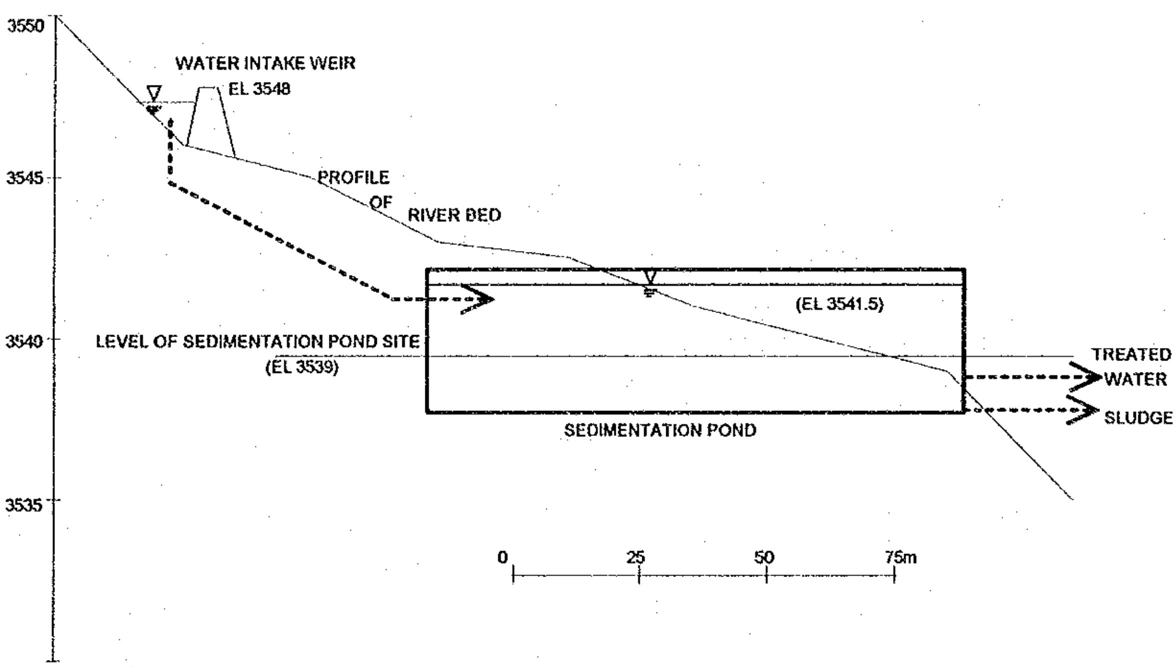


FIGURE 4 LONGITUDINAL PROFILE OF SEDIMENTATION POND

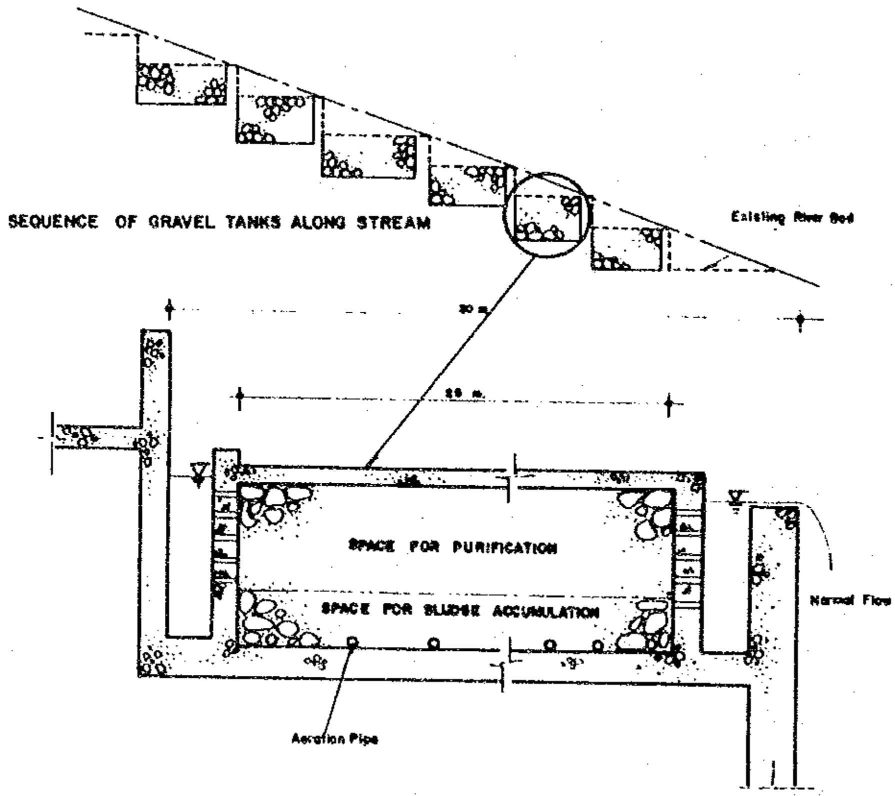


FIGURE 5 SCHEMATIC DRAWING OF GRAVEL CONTACT PURIFICATION

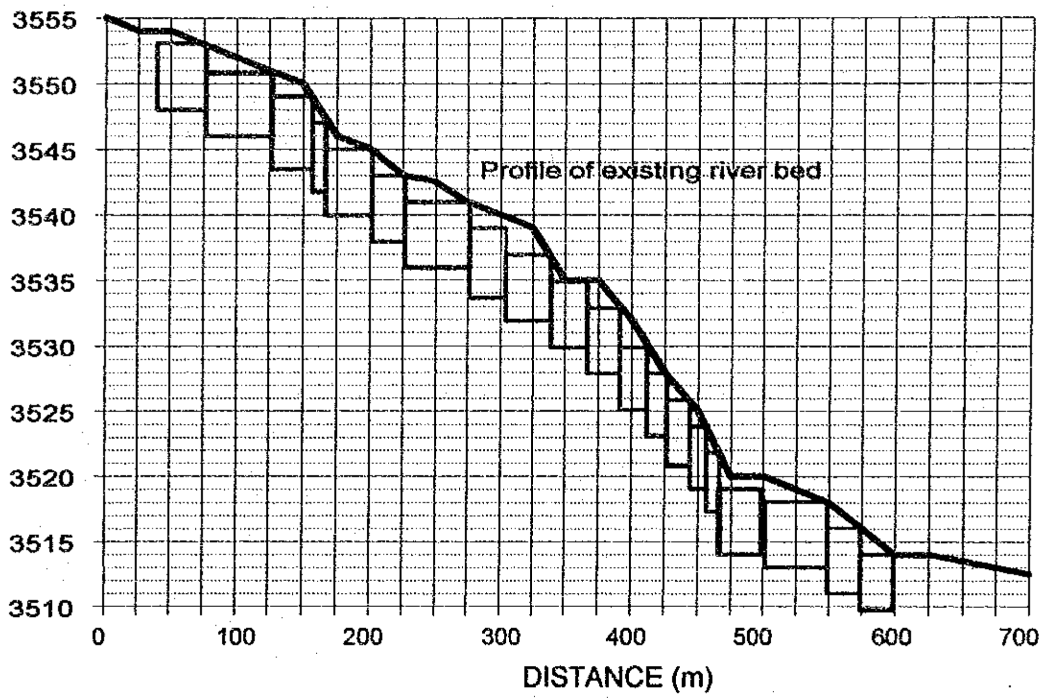


FIGURE 6 LONGITUDINAL PROFILE OF GRAVEL CONTACT TANKS

APPENDIX-C

POSSIBILITIES OF LAND ACQUISITION FOR WASTEWATER TREATMENT SITES NECESSARY TO TREAT ALL WASTEWATER FROM THE CENTRAL ZONE

1 INTRODUCTION

A decentralized plant option was eliminated from further consideration as an alternative for the Basic Plan as a result of investigation based on information provided by SAMAPA on available lands for wastewater treatment plants. Further investigations were requested by the Bolivian side in the Meetings held between 26th and 29th, October, 1992, to study the possibilities of acquiring lands in the Central zone necessary to treat all the wastewater from that zone.

This Appendix presents the sites necessary to locate small wastewater treatment plants in the city area so that all wastewater from the Central zone (excluding the Orkojahuira catchment) could be treated by small plants scattered in the city area, and the results of the evaluation on the possibilities of acquiring those sites.

2. PROPOSED PLANT SITES

2.1 Selection of Plant Sites

Sewer collection networks are spread throughout the urban areas. Most of these networks have outlets to small rivers which flow into the Choqueyapu river. Therefore, it is natural to regard the catchment of each small river as a minimum sewer division for one treatment plant.

Sewage is usally collected by gravity flow to a plant site, to avoid the capital and operating costs of lift stations, as long as topographical conditions of the area allow it. This is very important factor in locating wastewater treatment plants in La Paz due to the extremely steep topographical conditions. Therefore, the preferred location for a plant site would be the most downstream point of each sewer division.

Another important factor for site selection is the slope of the land. Wastewater treatment facilities/equipment should be placed on the flat land. Although it would be possible to level steeply sloping land, it is not practical in the concerned area because it would usually affect a larger area than the area actually necessary for a plant construction.

Accordingly, the sites for each sewer division were selected from relatively flat land located in the vicinity of the Choqueyapu river. The sewer divisions and the selected locations of all plants are presented in Figure 1 of this Appendix and the detailed locations of each site are shown in Figure 2 to Figure 7 of this Appendix.

2.2 Characteristics of the Selected Sites

The required area and present landuse of each site are as shown in Table 1.

TABLE 1 AREA REQUIREMENT AND PRESENT LANDUSE OF SELECTED SITES

No. of Plant *	Sewer Divisions *	Amount of Wastewater Generated (m3/day)	Required area (m2) **	Type of Landuse	Property	Major Buildings
1	A1	5.084	6,930	Industrial	Private	Industrial Buildings
2	A2	2.057	3,730	Educational	Public	School buildings
3	A3	2.133	3,770	SAMAPA	Public	Site of WTP
4	A4	1.133	2,420	Industrial	Private	Factory buildings
5	A5, A6	3.619	5,460	Industrial	Public	Municipality Slaughter house
6	B1	2.771	4,530	Industrial	Private	Beer factory
7	B2, B3	29.094	23,500	Housing / Commercial	Public/ Private	Lanza market, Commercial Buildings
8	B4	18.401	17,050	Housing / Commercial / Offices	Public/ Private	Cinema, Banks
9	B5	2.725	4,480	Industrial	Private	No building
10	B6	5.204	7,040	Housing / Cultural / Commercial	Public/ Private	The Culture House of La Paz, Ismar Building
11	B7, B8	10.372	11,410	Recreational	Public	Plant Nursery
12	C1, C2	24.243	20,680	Housing / Educational / Commercial	Public/ Private	University UMSA
13	C3	9.109	10,420	Housing / Commercial	Private	Spanish embassy, Hospital
14	C4	9.886	11,040	Recreational	Private	Labor Union Soccer Field
15	D	16.596	15,860	Housing	Public/ Private	Police School, Housing Building

NOTE: * For locations, refer to Figure 1 of this Appendix

** Area requirement (sq.m.) = 2.220 x (Wastewater, cu.m./day)^0.7

3. EVALUATION OF POSSIBILITIES TO ACQUIRE PROPOSED PLANT SITES

Judging from the present landuse of the selected plant sites shown in Table 1, a great difficulty in land acquisition is foreseen except for a few sites. Most of the sites are occupied with buildings which form the well established commercial and office area of La Paz. The relocation of those buildings could not be done without considerable changes in the urban planning. The land price alone for the selected sites is expected to be very high. However, the required costs for land acquisition of the selected sites would be much higher than the land price because they would include the cost necessary for the relocation of such buildings.

The results of an evaluation on the possibilities of land acquisition performed by HAM personnel shown in Table 2 seems to reflect this situation. They have judged that three sites could be acquired. One of them is a Minucipality's slaughter house which is now planned to be relocated. The other two are a nursery and a football field, having no buildings. Nine sites among the fifteen were judged to be impossible to acquire and the remaining three were judged to be possible to acquire but with great difficulty.

Both the study team's observation and the evaluation by HAM indicate a difficulty in land acquisition for necessary sites to treat all wastewater by small treatment plants. Therefore, the decentralized plant option is eliminated as an alternatives for the Basic Plan.

TABLE 2 EVALUATION OF POSSIBILITIES OF LAND ACQUISITION BY HAM

No. of Plant	Sewer Divisions	Possibility of Land Acquisition
1	A1	Impossible
2	A2	Possible but difficult
3	A3	Possible but difficult
4	A4	Impossible
5	A5, A6	Possible
6	B1	Impossible
7	B2, B3	Impossible
8	B4	Impossible
9	B5	Possible but difficult
10	B6	Impossible
11	B7, B8	Possible
12	C1, C2	Impossible
13	C3	Impossible
14	C4	Possible
15	D	Impossible

Even if it were possible to acquire the required sites, and without even considering the extremely high cost of such sites, the estimated construction costs and other comparisons between a centralized plant option and a decentralized plant option, shown in Table 3, indicate the great economic disadvantage of the decentralized plant alternative.

TABLE 3 COST COMPARISONS BETWEEN CENTRALIZED OPTION AND DECENTRALIZED OPTION

	Treatment Capacity (m3/day)	Required Area (m2)	Construction Cost (1000US$) Plant	Pipe/Weir	Total	Operation/maintenance Cost (1000US$/year)
Centralized Option	**176,987**	**83,160**	**88,884**	**8,846**	**97,730**	**3,427.0**
Decentralized Option	**142,427**	**148,320**	**146,046**	**0**	**146,046**	**6,161.4**
Plant-1	*5,084*	*6,930*	*6,658*	*0*	*6,658*	*288.6*
Plant-2	*2,057*	*3,730*	*3,439*	*0*	*3,439*	*153.6*
Plant-3	*2,133*	*3,770*	*3,532*	*0*	*3,532*	*157.5*
Plant-4	*1,133*	*2,420*	*2,225*	*0*	*2,225*	*101.4*
Plant-5	*3,619*	*5,460*	*5,195*	*0*	*5,195*	*227.8*
Plant-6	*2,771*	*4,530*	*4,352*	*0*	*4,352*	*189.1*
Plant-7	*29,094*	*23,500*	*23,790*	*0*	*23,790*	*973.5*
Plant-8	*18,401*	*17,050*	*17,028*	*0*	*17,028*	*707.5*
Plant-9	*2,725*	*4,480*	*4,223*	*0*	*4,223*	*186.8*
Plant-10	*5,204*	*7,040*	*6,773*	*0*	*6,773*	*293.4*
Plant-11	*10,372*	*11,410*	*11,205*	*0*	*11,205*	*474.4*
Plant-12	*24,243*	*20,680*	*20,824*	*0*	*20,824*	*857.3*
Plant-13	*9,109*	*10,420*	*10,192*	*0*	*10,192*	*433.4*
Plant-14	*9,886*	*11,040*	*10,818*	*0*	*10,818*	*458.8*
Plant-15	*16,596*	*15,860*	*15,792*	*0*	*15,792*	*658.3*

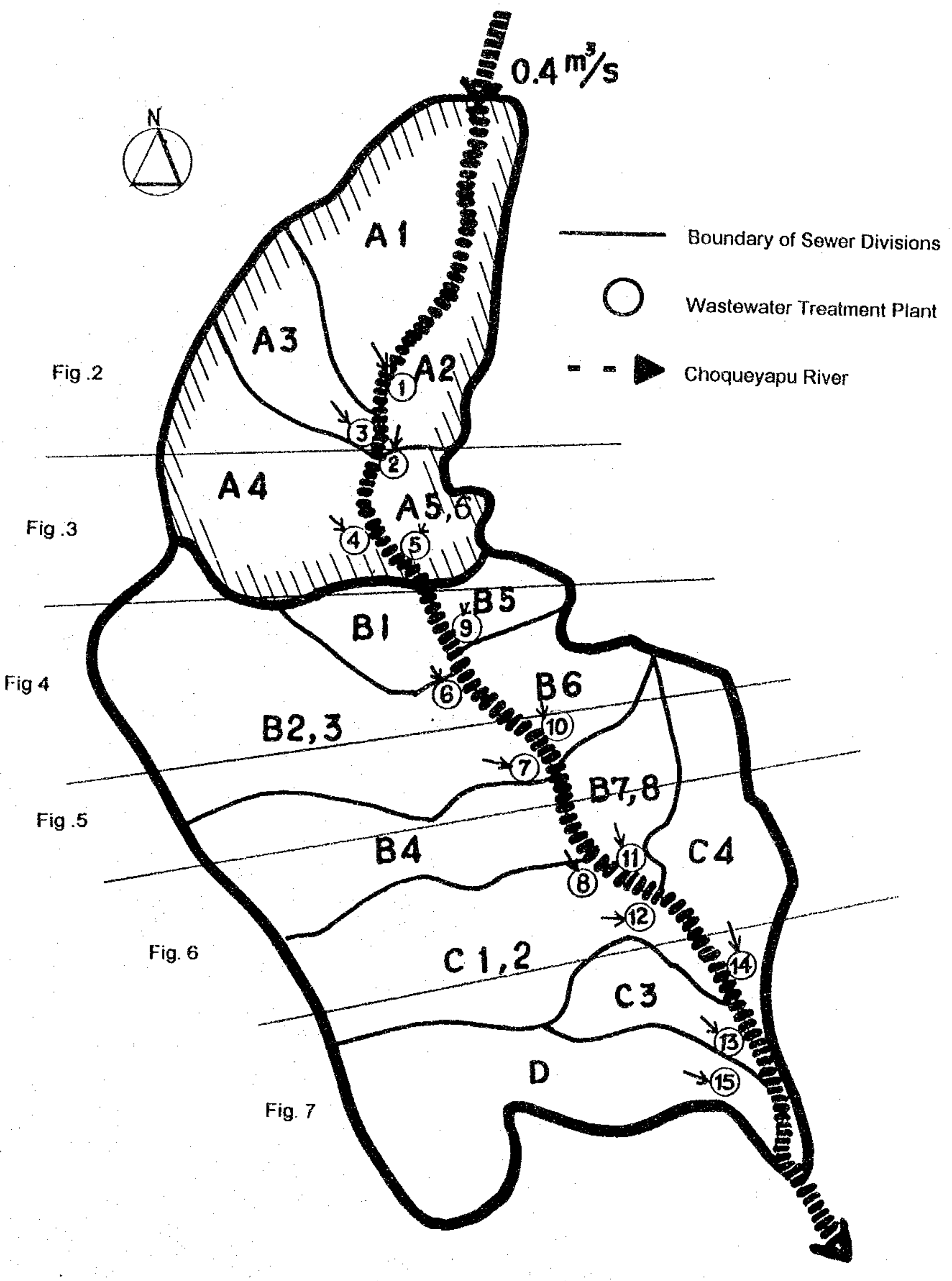


FIGURE 1 SEWER DIVISIONS AND LOCATION OF PLANT SITES

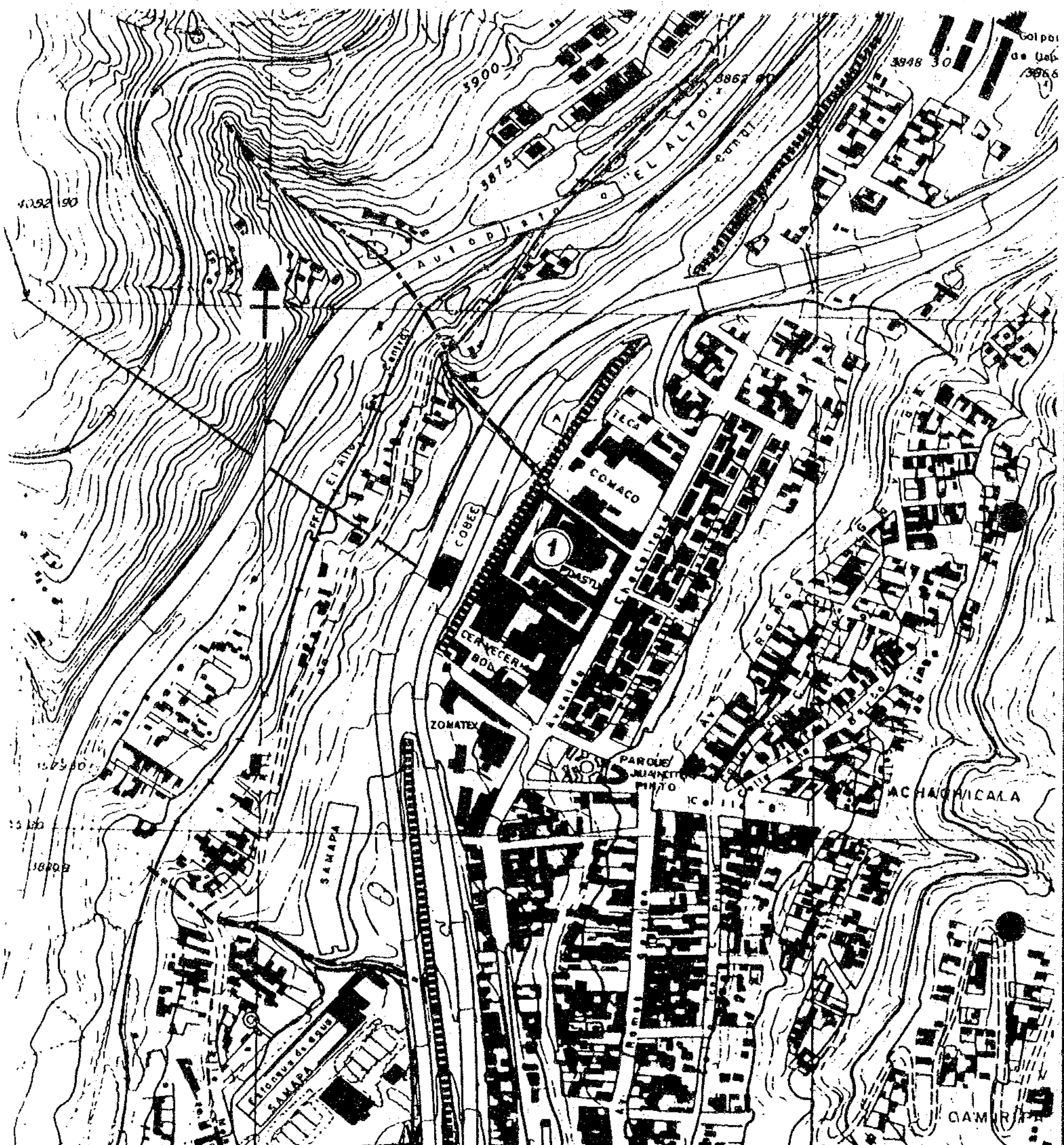


FIGURE 2 Location of Plant Sites (1, 3)

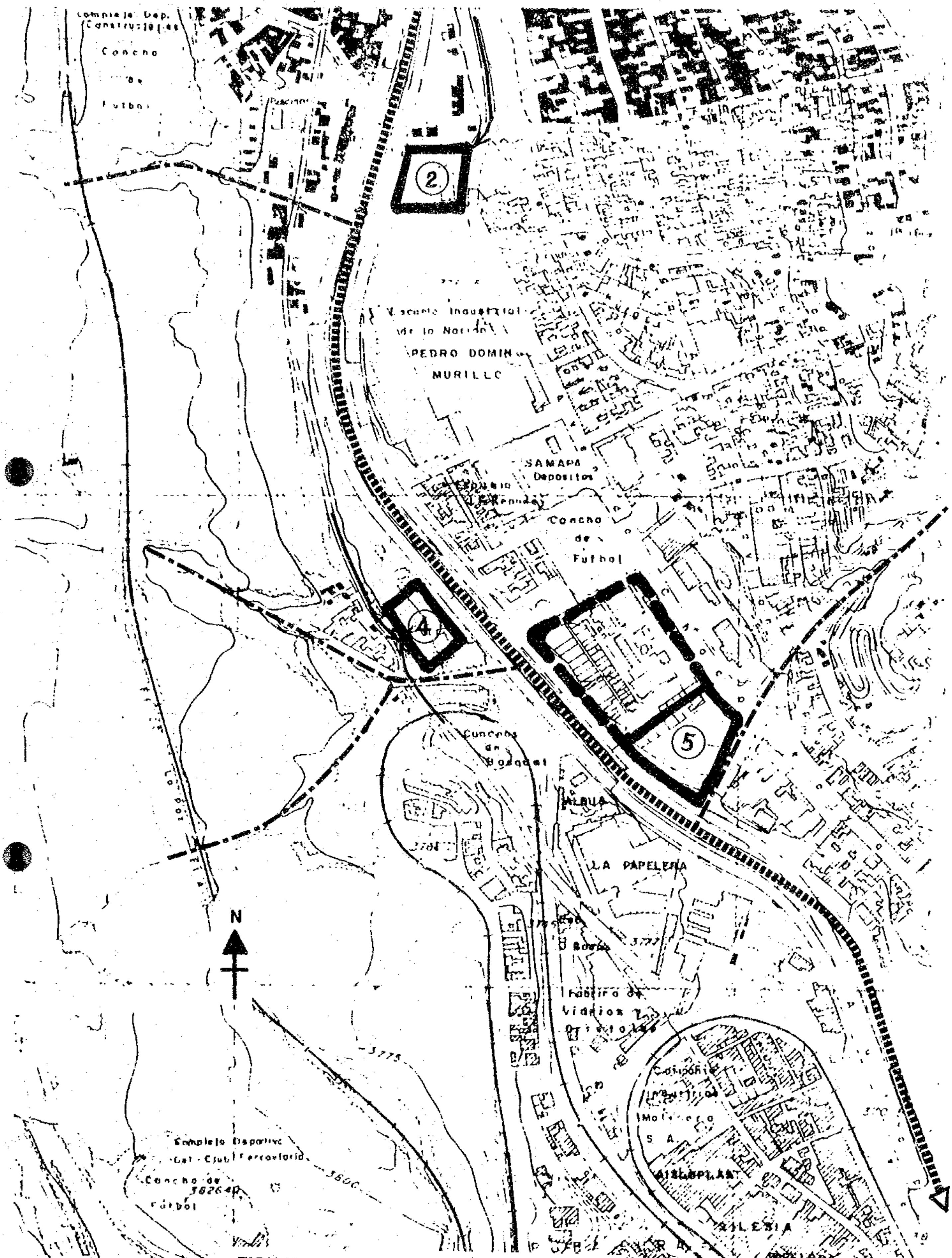


FIGURE 3 Location of Plant Sites (2, 4, 5)

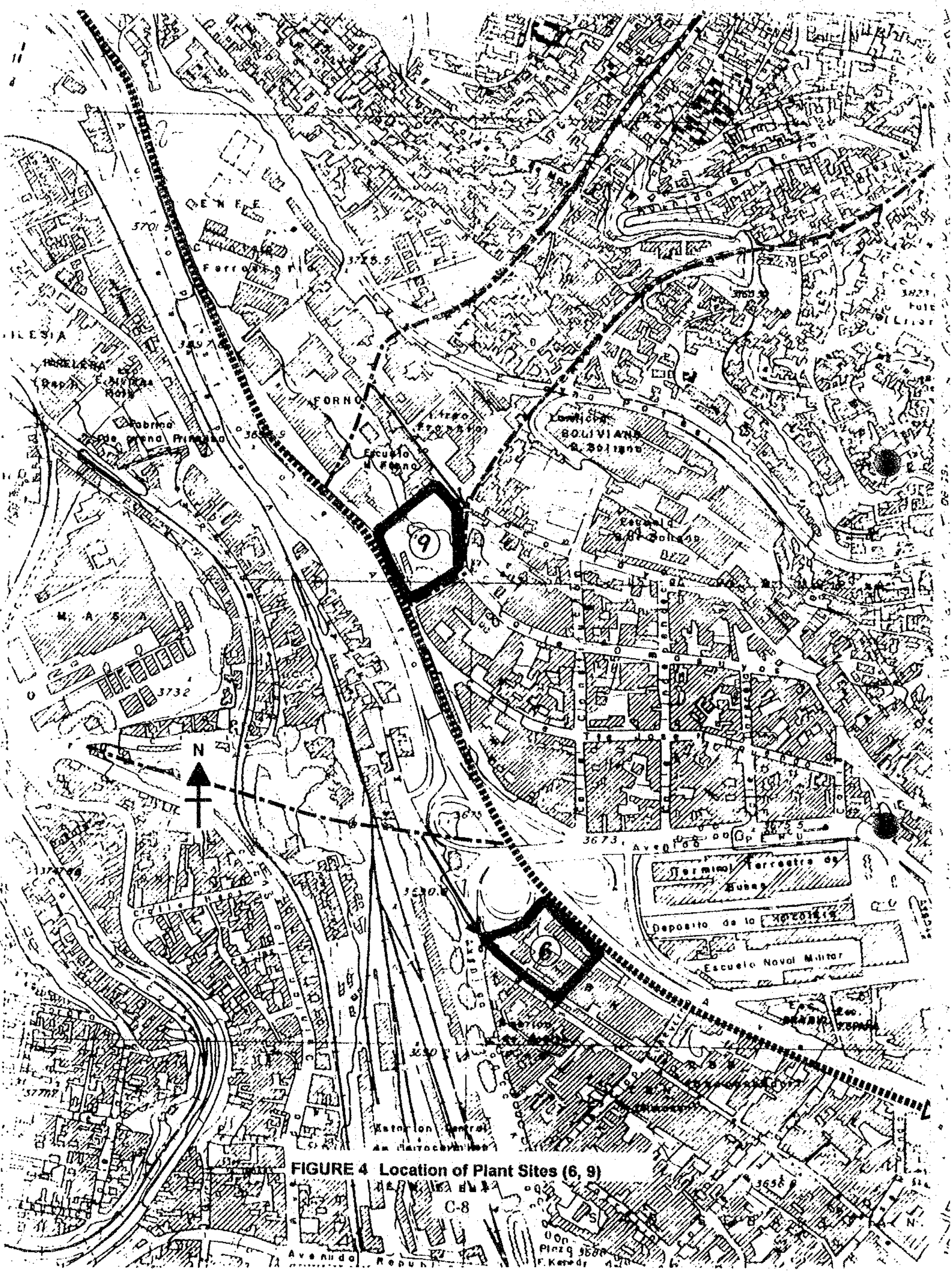


FIGURE 4 Location of Plant Sites (6, 9)

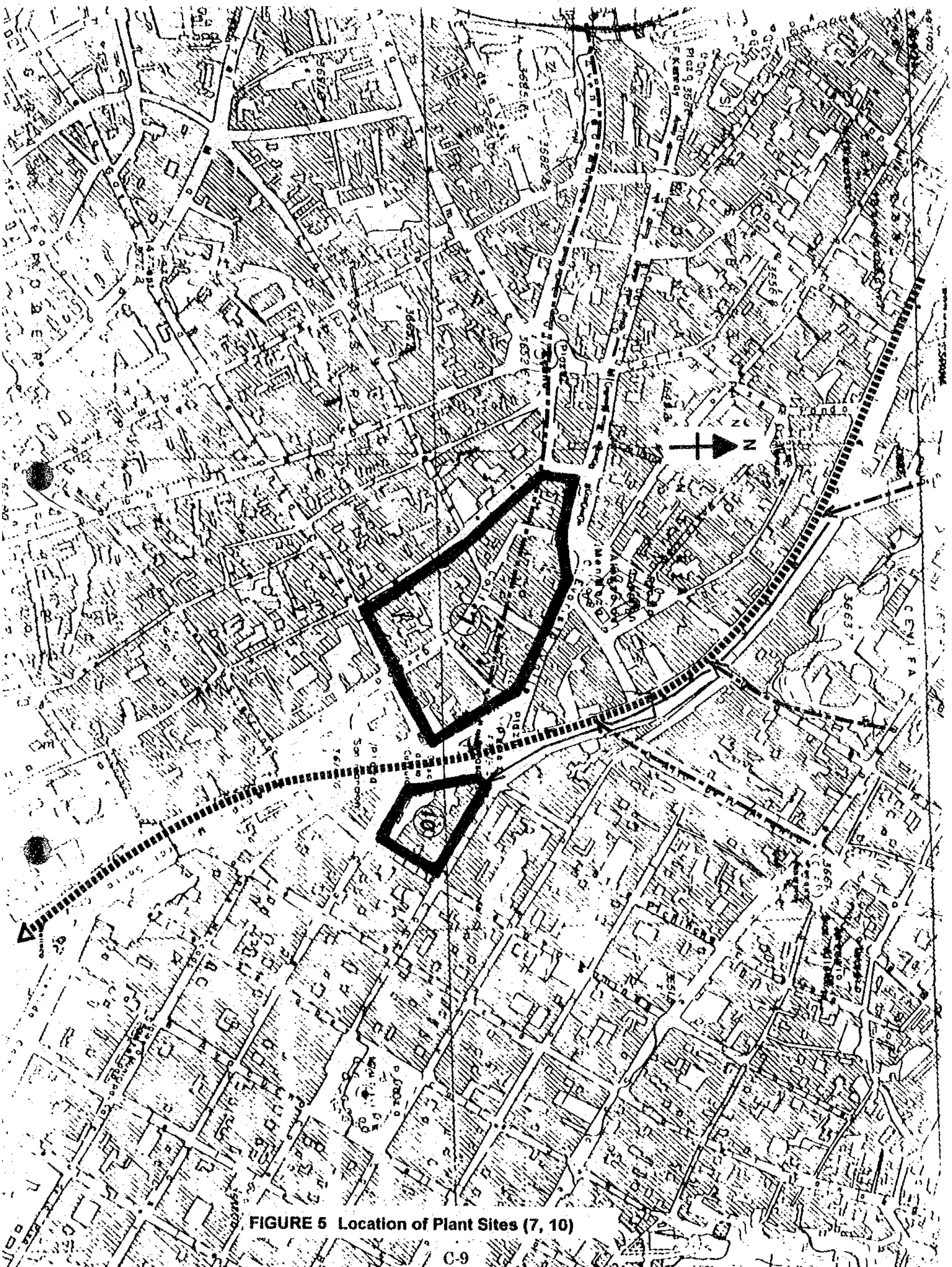


FIGURE 5 Location of Plant Sites (7, 10)

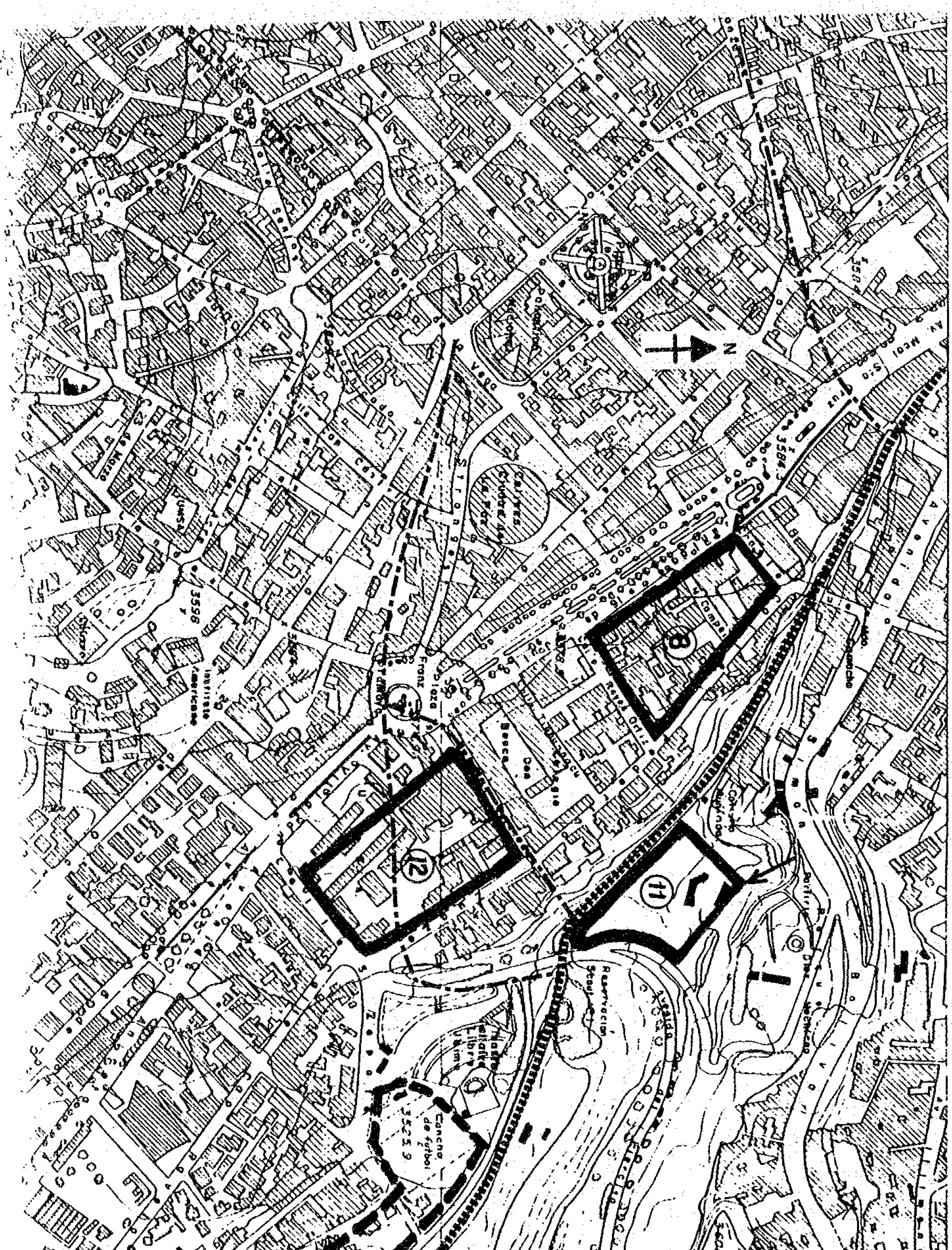


FIGURE 6 Location of Plant Sites (8, 11, 12)

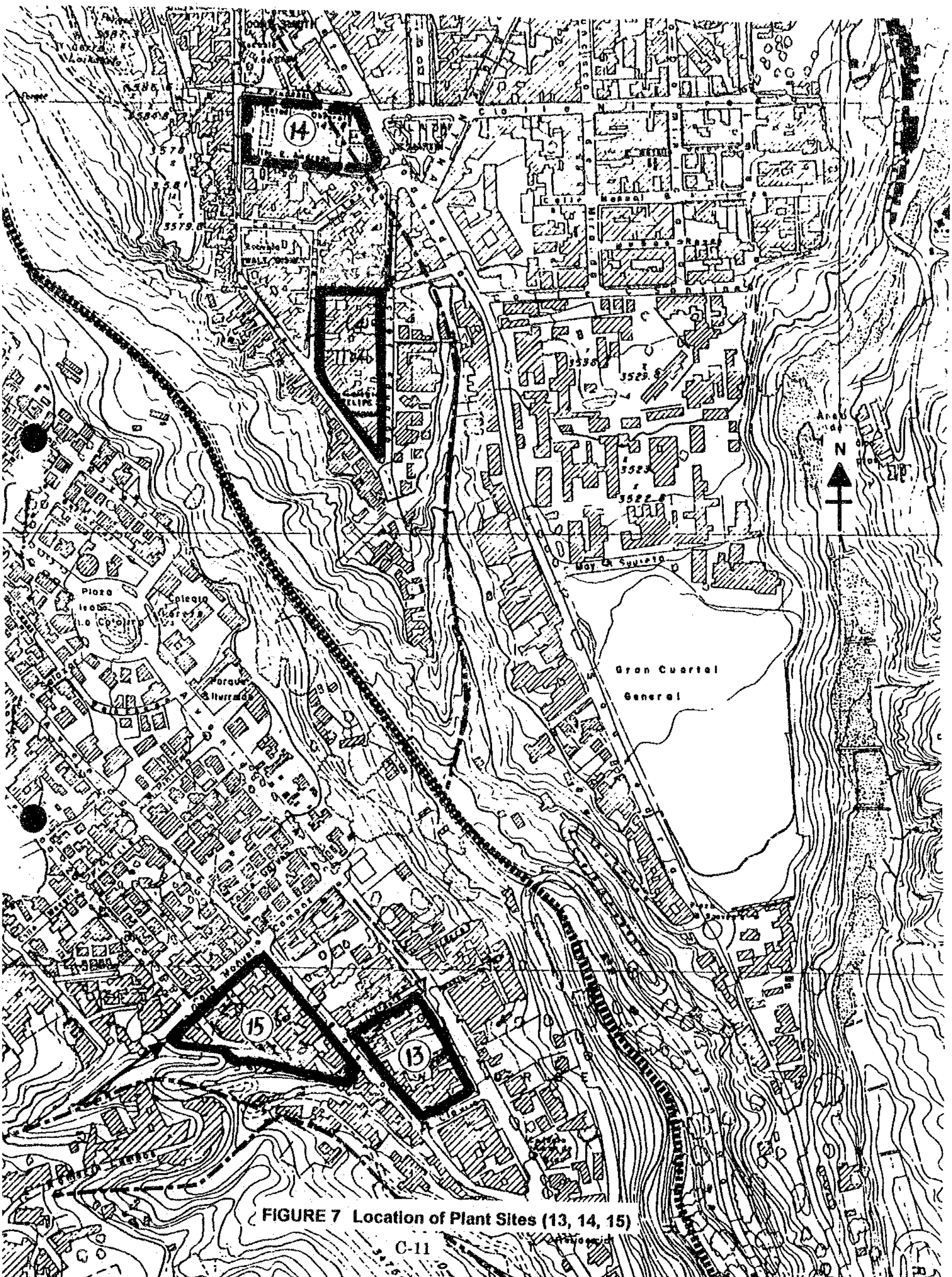


FIGURE 7 Location of Plant Sites (13, 14, 15)

APPENDIX D

PRELIMINARY DESIGN OF WASTEWATER TREATMENT PLANT (Aerated Lagoons for Lipari Option)

1. Introduction

Wastewater characteristics and design flows are described in Chapters 4 and 5 of the report. The wastewater treatment process for this project was selected based on a preliminary screening, economic comparison and overall evaluation of four alternatives, as presented in Chapter 5. The overall treatment plant, consisting of aerated lagoons and related facilities, is described in this Appendix. The priority project, land acquisition requirements and project implementation are described in Chapter 7 of the main body of the report.

2. Site Preparation

The sites chosen for the lagoons and related treatment facilities are located along the Choqueyapu River bed in the flood plain as shown in Figure 5.2.7. The selected sites will be elevated above the flood level and protected by gabion revetments along the river. On the basis of flood studies, prepared by the Bolivian Counterpart, the height of the revetments (gabions) is 3 m (See Figure 7.4.3). The general finished grade elevation of the plant sites will be also about 3 m above the river bed to minimize flooding problems, and preclude the need for flood water pumping (See Figure 7.4.3). Plant Sites #1 and #2 are influenced by steep streams (quebradas) which intersect them and reduce the areas available for construction. Drainage canals will be required for these two sites.

3. Preliminary Treatment

Preliminary treatment works at the inlet of the plant, consisting of bar screens, grit chambers, and flow measuring devices are proposed for this project. The bar screens and grit chambers are for the following purposes:

- Removal of grit, heavy solids, and floating materials
- Protect piping and valves from damage by clogging and abrasion
- Removal of large particulate matter, reducing the load and volume requirements for subsequent processes

The flow metering device (flume and measuring equipment) is to provide flow information that will be useful for operations, for assessing design and operational changes, and for establishing future design criteria. The range of design criteria and the values adopted for the grit chambers of this project are summarized as follows:

Type of Grit Chambers - Horizontal Flow

Preliminary Design Criteria
(Based on Average Flows)

	Range Ref. E7	This Project
Detention time (minutes)	0.5 - 1.5	0.8 - 1.0
Water Depth (m)	0.6 - 1.5	0.7 - 1.0
Length (m)	3.0 - 25.0	18.0
Horiz. Velocity (m/s)	0.15 - 0.4	0.31 - 0.37

The proposed primary device for measuring flow is a Parshall Flume, commonly used in wastewater measuring applications because of its self cleaning characteristics. Two Parshall flumes would be provided in the initial stage, and these would be duplicated in a later stage.

It is proposed that two manually cleaned grit chambers be provided in the first stage to permit continuous operations normal operation at while one chamber is out of service for cleaning. Each basin will be designed for normal operation at 50% of the initial design capacity, but will be capable of operating at the peak hours rate. Provision will be made to duplicate these units in a later stage; refer to Figure 7.4.5.

The depth in the grit chambers would be controlled by Parshall Flumes; see Figure 7.4.2. Each Parshall Flume would be designed for the ultimate flow (peak hourly rate in the year 2010, with 3 of the 4 basins operating), although it would normally operate at 140,000 / 2 = 70,000 m^3/day in the first stage and 230,000 / 4 = 57,500 m^3/day in the final stage. The operating depth in the flume at these flows is between 0.3 and 0.35 m. The bottom of the grit chambers is set at 0.5 meters below the bottom of the flume to provide velocities of approximately 0.35 m/sec. under normal flow conditions.

On the basis of the criteria adopted for these facilities, assuming a chamber width of 2.665 meters (same as the Parshall Flume entrance width), the volume of each chamber under initial conditions would be 18 • 0.844 • 2.665 = 40.5 cu.m.. In the first stage, with both basins in service, the average detention time would be 40.5/(140,000 • 0.50 / 1440) = 0.83 minutes and the average horizontal velocity would be (140,000 • 0.50) / (86,400 • 2.665 • 0.844) = 0.36 m/sec. Under final flow conditions, with all grit chambers operational, the average detention time would be 18.00 • 0.804 • 2.665 • 4 / (230,000 / 1440) = 0.97 minutes, and the average horizontal velocity would be (230,000) / 86,400 / (4 • 2.665 • 0.804) = 0.31 m/sec.

Preliminary design criteria for the bar screens are summarized as follows:

Bar Size:	1 x 5 cm
Spacing:	5 cm
Slope:	45^0 from Vertical
Ave.Velocity:	0.5 - 0.9 m/sec (through the screen)
Cleaning:	Manual

The preliminary layout is shown in Figure 7.4.1, and preliminary design sketches are shown in Figure 7.4.2 and 7.4.5.

Preliminary cost estimates are presented in Section 8 of this Appendix.

4. Aerated Lagoons

4.1 General

The proposed lagoons are to be aerated by mechanical means. The preliminary design for ultimate capacity is based on completely-mixed aerated lagoons. Partially mixed aerated lagoons, which require considerably less power to operate would require more than twice as much area as the completely mixed lagoons. Preliminary design criteria used for the completely mixed aerated lagoons are as follows:

Preliminary Design Criteria

Flow = 230,000 cu.m./day
S_0 = Influent BOD = 250 mg/l
Influent suspended solids = 250 mg/l
S = Effluent BOD = 50 mg/l (80% reduction)
Effluent suspended solids = 70 mg/l (70% reduction)
Design (Winter) Air temp = waste temp = 8.8 ^oC
(Summer = 11.5 ^oC)
K_{20} (First order soluble BOD removal rate constant) = 2.5 / day
Elevation = 3080 m
Aeration Constants α = 0.9, β = 1.0
Oxygen concentration to be maintained = 1.5 mg/l
Lagoon Depth = 6 m (max.); 4.5 m (average)
Design mean cell-residence time = 3 days
Other kinetic coefficients: Y = 0.65, k_d = 0.07
Side Slopes : Exterior 3:1; Interior 2:1

Note: for final design, the removal rate constant k_{20} and other kinetic coefficients should be evaluated in bench scale tests; see Reference E4, Appendix H

On the basis of these criteria, the lagoon dimensions (for the year 2010) are determined as follows:

Volume = 230,000 · 3 = 690,000 cu.m.
Surface Area = 690,000 / 4.5* = 153,333 m^2 = 15.33 Ha

(Note) * 4.5m = average depth of lagoons over surface area

The corrected BOD removal rate constants (winter and summer time) are estimated (using Eq. 8-14, Ref. E4) as follows:

$k_T / k_{20} = q^{T-20}$, where q = Temperature Activity Coefficient = 1.06
from which $k_{8.8} = 2.5 \cdot (1.06)^{8.8-20} = 1.30$

and $k_{11.5} = 2.5 \cdot (1.06)^{11.5-20} = 1.52$

The BOD at the effluent of the aerated lagoons is then estimated (using Eq. 10-20 Ref. E4) as follows:

$S/So = 1/(1+k_T \cdot t)$,

where, S = effluent BOD, So = influent BOD, and

t = design residence time (days)

From this equation:

$S = 250/(1+1.3 \cdot 3) = 51$ mg/l (Winter), and

$S = 250/(1+1.52 \cdot 3) = 45$ mg/l (Summer)

The concentration of biological solids produced is estimated (using Equation 8-27 of Ref. E4) as follows:

$X = Y \cdot (So - S) / (1+k_d \cdot t)$

$0.65 \cdot (250\text{-}51) / (1+0.07 \cdot 3) = 106.9$ mg/l VSS (Winter), and

$0.65 \cdot (250\text{-}45)/(1+0.07 \cdot 3) = 110.1$ mg/l VSS (Summer)

The estimated suspended solids in the lagoon effluent before settling is then:

$SS = 250 + 106.9 / 0.8 = 383.6$ mg/l (Winter), and

$SS = 250 + 110.1 / 0.8 = 387.5$ mg/l (Summer)

With the low overflow rate in the large sedimentation basins (1 day detention; see Section 5 of this Appendix.), a suspended solids concentration of about 70 mg/l should be obtained.

The oxygen requirements at design flow are estimated (using Eq. 10-5 of Ref. E4) as follows:

O_2 (kg /day) = $Q \cdot (So\text{-}S) / (10^3 \cdot f) - 1.42 \cdot (P_x)$

where, f = conversion factor for converting BOD_5 to BOD_L, and

P_x = mass of organisms wasted

P_x is estimated (from the VSS calculated earlier), as follows:

$106.9 \cdot 230{,}000 / 10^3 = 24{,}587$ kg/day (Winter), and

$110.1 \cdot 230{,}000 / 10^3 = 25{,}323$ kg/day (Summer)

The estimated oxygen requirements for the aerated lagoons are then:

$230{,}000 \cdot (250 - 51) /(10^3 \cdot 0.68) - 1.42 \cdot 24{,}587))$= 32,395, kg/day (Winter),

and $230{,}000 \cdot (250 - 45) /(10^3 \cdot 0.68) - 1.42 \cdot 25{,}077)$= 33,728 kg/day (Summer)

The ratio of oxygen required to BOD removed (Summer) is 33,728 / ((250-45) • 230,000/10^3) = 0.72

4.2 Aeration Equipment

Using a field transfer rate of 0.6 kg O_2 / Hp • hr, the horsepower requirements for oxygen transfer is,

$$(33{,}720 / 24)\ (1 / 0.6) = 2{,}342 \text{ Hp}$$

However, the controlling factor for the power requirement (for complete mixing of domestic wastewater) is that required for mixing. For complete mixing, the hp requirement may be much more than that required for aeration. For a complete mixed-flow regime, the power requirement (on the basis of criteria from several reference sources) ranges from 3 to 15 kW per 1000 cu.m. of lagoon volume. The range of estimated power requirements, based on these references, then varies from 2070 to 10,350 kW.

The aerators recommended for the project are float-mounted surface aerators with draft tubes similar to those shown in Figure D- 6. The aerators would be moored to the shore to facilitate maintenance and repair. In the design of aeration equipment for lagoons, it is common practice to use operational factors determined by equipment manufacturers for specific aeration and mixing equipment. One manufacturer offers 75 Hp units with draft tubes having a zone (diameter) of complete aeration of 116 m and a zone of complete mixing of 40 meters. For these units the required number of units varies from 2 x 12 = 24 in the first stage (partial mix) to 5 x 16 = 80 in the last stage (complete mix). The total power requirements for these units would vary from 24 x 75 = 1800 Hp in the first stage to 80 x 75 = 6000 Hp in the final stage. These units would be furnished in stages to provide for increasing flows and removal efficiencies. The layout of the aeration units in a typical lagoon for the first and final design stages are shown in Figure D-7.

The actual operation of the aerators will depend on treatment results. It is expected that the above horsepower requirements will be fully utilized only during critical low flow periods. The use of complementary mechanical mixers will be considered, in the final design, in order to reduce the total operating horsepower.

4.3 Layout of Lagoons

Preliminary layouts of the aerated lagoons, as they are to be constructed in the first stage, are presented in Figures 7.4.1 to 7.4.6. The last cell in each series would be converted to two cells in the second stage construction by adding intermediate dikes; See Figure D-1.

Inlet works, between the preliminary treatment works and the lagoons will consist of a distribution channel leading to valved pipes entering each lagoon in two locations; see Figure 7.4.5 .

To facilitate the maintenance of constant levels in the lagoons, in view of the elevation difference between succeeding lagoons, interconnections between lagoons and outlet works will be controlled by two weirs for each lagoon. These weirs will be connected to corrugated metal pipes; see Figure D-2. If it is desired to bypass a particular cell or cells, the flow can be temporarily diverted to the opposite set of cells and specially made plugs can be attached to the end of the metal pipes; see Figure D-3. To permit draining of the cells, there will also be a valved pipe near the bottom connecting each cell to a lower cell or to the river. This pipe could also be used during normal operations to improve flow patterns. The effluent from the last cells will be discharged to the sedimentation basins, and then to a channel leading to the river.

5. Solids Separation Basins

The suspended solids concentration leaving the completely-mixed aerated lagoons will be at a high level due to the mixing effects of the aerators. Large earthen sedimentation basins are proposed to reduce the suspended solids content, as well as to provide sludge storage and additional digestion.

The following are the major considerations in this design:

- detention time adequate for solids separation
- sufficient volume of sludge storage
- minimize algal growth
- control of odors from anaerobic sludge

Algal growths can be controlled by limiting the hydraulic detention time to a maximum of 2 days and odors can be controlled by providing a minimum depth of 1 m over the sludge deposits.

For the proposed sedimentation basins, the following criteria are proposed:

- Sedimentation detention time of 1 day
- Liquid layer above the sludge level = 2.0 m
- Basins will be cleaned (pumped out) every 4 years

The mass of sludge that will accumulate in the sedimentation basin each year (not considering anaerobic digestion) is estimated (from Ref. E4, page 612) as follows:

Mass = $[(SS_i - SS_e) / 10^6] \cdot Q \cdot 365 = [(385 - 70) / 10^6] \cdot 230{,}000 \cdot 365$

= 26,444.2 tons/year

The mass of volatile solids added per year, assuming that VSS = 70% of SS, is:

VSS = $26{,}444.2 \cdot 0.7$ = 18,510.9 tons/year

The mass of fixed solids added per year, assuming that VSS = $0.7 \cdot$SS, is:

FS = 26,444.2 -18,393.4 = 8050.8 tons/year

It is assumed (from Ref. E4, page 613) that the maximum volatile solids reduction is 75%, and that this will occur for all the sludge accumulated except that deposited during the last year. Simplifying that the deposited volatile solids undergo a liner decomposition, the following equation (from Ref. E4, page 613) can be used to approximate the amount of volatile solid accumulated at the end of a given year:

$$VSS_t = [0.7+0.25 \cdot (t-1)] \cdot 18{,}510.9$$

where t = time, yr.

For a 4 year period , VSS = $[0.7+0.25 \cdot (4 - 1)] \cdot 18{,}510.9$ = 26,840.8 tons.

For the same 4 year period the mass of fixed solids would be

$4 \cdot 8050.8$ = 32,203.2 tons

The total solids mass deposited in the sedimentation basin during the four year period would then be 26,840.8 + 32,203.2 = 59,044 tons

The required liquid volume and dimension for the sedimentation portion of the basins:

$$\text{Vol.} = 1 \text{ day} \cdot 230{,}000 \ m^3 / \text{day} = 230{,}000 \ m^3$$

$$\text{Surface Area} = 230{,}000 \ m^3 / 2.0 \ m = 115{,}000 \ m^2 = 11.5 \ \text{Ha}$$

The design overflow rate for such a configuration is 230,000 / 115,000 = 2 m/day, which should provide a SS removal of over 80% and a final SS of about 70 mg/l.

The required depth for storage of sludge is determined as follows:

The accumulated mass of sludge over the surface area = 59,044/115,000 = 0.5134 tons / m2. Assuming that the deposited sludge will compact to an average concentration of 15% solids and that the density of the sludge is 1.06, the average depth is determined as follows:

$$0.5134 / d = 1.06 \cdot 0.15, \quad d = 0.5134 / (1.06 \cdot 0.15) = 3.23 \ m$$

The total average depth of the sedimentation basins would then be 2.0 + 3.23 = 5.23 m.

Allowing for the lagoon slope of 2:1, the required maximum depth is about 6 meters. It is proposed that there be four parallel cells, located at Site 2, each with a surface area of approximately 3 hectares; see Figures D-4 and D-5.

A preliminary cost estimate is presented in Section 8 of this Appendix.

6. Sludge Storage Basin

During the basic plan stage, it was first proposed to provide sludge storage basins for additional drying, digestion and concentration of the sludge from the sedimentation basins. The justification for these basins was to reduce the cost of hauling sludge to the landfill (or other land application) by increasing the solids content. However, it was determined later that the added land acquisition, capital and operating costs for such storage basins considerably exceed the savings in sludge hauling costs. The sludge storage basins were therefore eliminated from further consideration. However, the municipality may wish to consider such basins at the landfill site for ultimate disposal or concentration of the sludge.

The sludge from the sedimentation basins will therefore be transported directly from the sedimentation basins to the landfill during all stages.

As mentioned earlier, another possibility to be investigated is the application of the liquid sludge from the sedimentation basins to agricultural or forest lands. When applied to such lands, the sludge acts as a soil conditioner, to increase water retention and soil tilth. It also provides valuable nutrients, in lieu of other sources of fertilizers. Careful study of the type of application and sludge content is important, however, because certain soil and crop types are to be avoided, and adverse constituents, particularly from industrial waste discharges, can be harmful to the soil. The potential for this type of application is great in the study area. However, extensive marketing and technical feasibility studies are required for such applications. It is therefore assumed for purposes of estimating operating costs, that the semi-dewatered sludge from the sedimentation basins, will be trucked to a municipal landfill site.

7. Hydraulics

Headlosses though the plant were calculated using the following formulas and values:

Pipe Flow

$H = f \cdot (L/d) \cdot (V^2/2g)$
$g = 9.10$
$H = (V/0.85 \cdot C \cdot R \cdot 0.63)^{1.85L}$
$C = 100$ to 140
$R = d/4$
$V = Q/(3.2416d^2/4)$

Open Channels

$H = (V \cdot n/R^{0.67})^2 \cdot L$
$n = 0.015$
$R = d \cdot w/(w+2d)$
Gate in Channel
$H = Q^2/(C \cdot A)^{2 \cdot 2g}$
$g = 9.8$

Weirs (Sharp Crested)

$H = (Q/C \cdot w \cdot L)$

Pipelines and open channels were sized to maintain minimum velocities of 0.6 m/sec and maximum velocities of 3 m/sec. Hydraulic gradients for the priority project are discussed in Section 7.4 of the report.

8. COST ESTIMATES - WASTEWATER TREATMENT PLANT

A preliminary cost estimate for the proposed aerated lagoons and related facilities of the wastewater treatment plant, to be constructed in three stages, is presented in the following Table D-1.

TABLE D - 1
SUMMARY OF PROJECT COSTS
WASTEWATER TREATMENT PLANT - LIPARI
(US$ Million in 1992 Prices)

	FIRST STAGE	SECOND STAGE	TOTAL PROJECT
Site Preparation			
Site#1	1.68		1.68
Site#2		0.79	0.79
Prelim. Treatment Works	0.38	0.38	0.76
Aerated Lagoons	3.75	1.74	5.49
Sedim. Basins (Site #1)	1.67		1.67
Sedim. Basins (Site #2)		1.32	1.32
Convert Sed. Basins (Site#1)		0.34	0.34
Canal from Site #1 to #2		0.43	0.43
Buildings	0.40		0.40
Acess Road Sites #1&2	0.28		0.28
Electrical	0.12	0.23	0.35
TOTAL CONSTRUCTION	8.28	5.23	13.51
Land Acquisition and R.O.W.	3.35	0.00	3.35
Engineering	0.83	0.52	1.35
Contingencies	1.16	0.52	1.69
TOTAL PROJECT COSTS (WWTP)	13.62	6.27	19.89

More detailed cost estimates for the first stage are presented in Section 7.4 of the report.

Operating costs were estimated for each of the stages and are summarized the following Table D-2

TABLE D-2
ESTIMATED OPERATING COSTS AERATED LAGOONS
(US$ In 1992 Prices)

	FIRST STAGE (140,000 m3/d)	SECOND STAGE (170,000 m3/d)	THIRD STAGE (200,000 m3/d)	FOURTH STAGE (230,000 m3/d)
Supervision				
Director	9,900	9,900	9,900	9,900
Operators/Engineers	14,520	21,780	21,780	21,780
Prelimin.Works				
Laborers	1,980	3,960	3,960	3,960
Aerated Lagoons				
Laborers	7,920	11,880	15,840	15,840
Night watch	1,650	1,650	1,650	1,650
Lab Technicians	14,850	19,800	19,800	19,800
Electric Technician	4,950	4,950	4,950	4,950
Mechanical Technician	4,950	4,950	4,950	4,950
Electric Power (for aeration)	347,654	1,359,704	1,599,652	2,454,026
Parts and Materials	10,000	29,048	48,152	67,200
Sedimentation Basins				
Drivers	3,960	5,940	7,920	7,920
Laborers	3,960	5,940	11,880	11,880
Nightwatch	1,980	1,980	1,980	1,980
Mechanic	2,250	4,500	4,500	4,500
Diesel Fuel *	5,976	11,952	20,005	28,058
Parts and Materials	1,500	3,000	4,000	5,000
Electric (for pumping)	235	235	393	551
Operations Building				
Secretary	4,620	9,240	9,240	9,240
Administrative Staff	4,620	9,240	9,240	9,240
Janitor	1,980	1,980	1,980	1,980
Electric	3,066	6,132	6,132	6,132
Supplies	1,500	1,500	1,500	1,500
Totals	444,121	1,519,361	1,799,504	2,682,137

*For Hauling Sludge
Assumes 10 km round trip

FIGURE D-1
PROFILE OF LAGOONS
SITE 1 - STAGE 2 WORKS

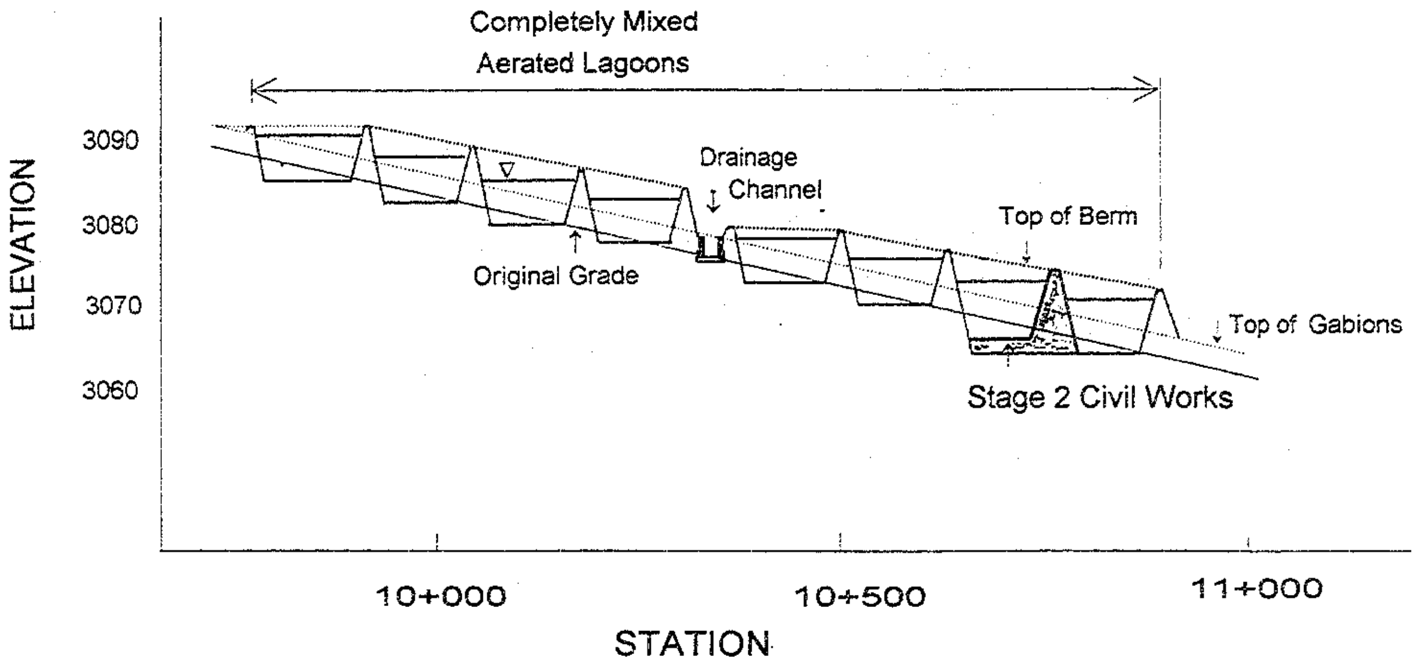


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FIGURE D-2
LAGOON CONTROL WORKS
SITE 1 - PRIORITY PROJECT

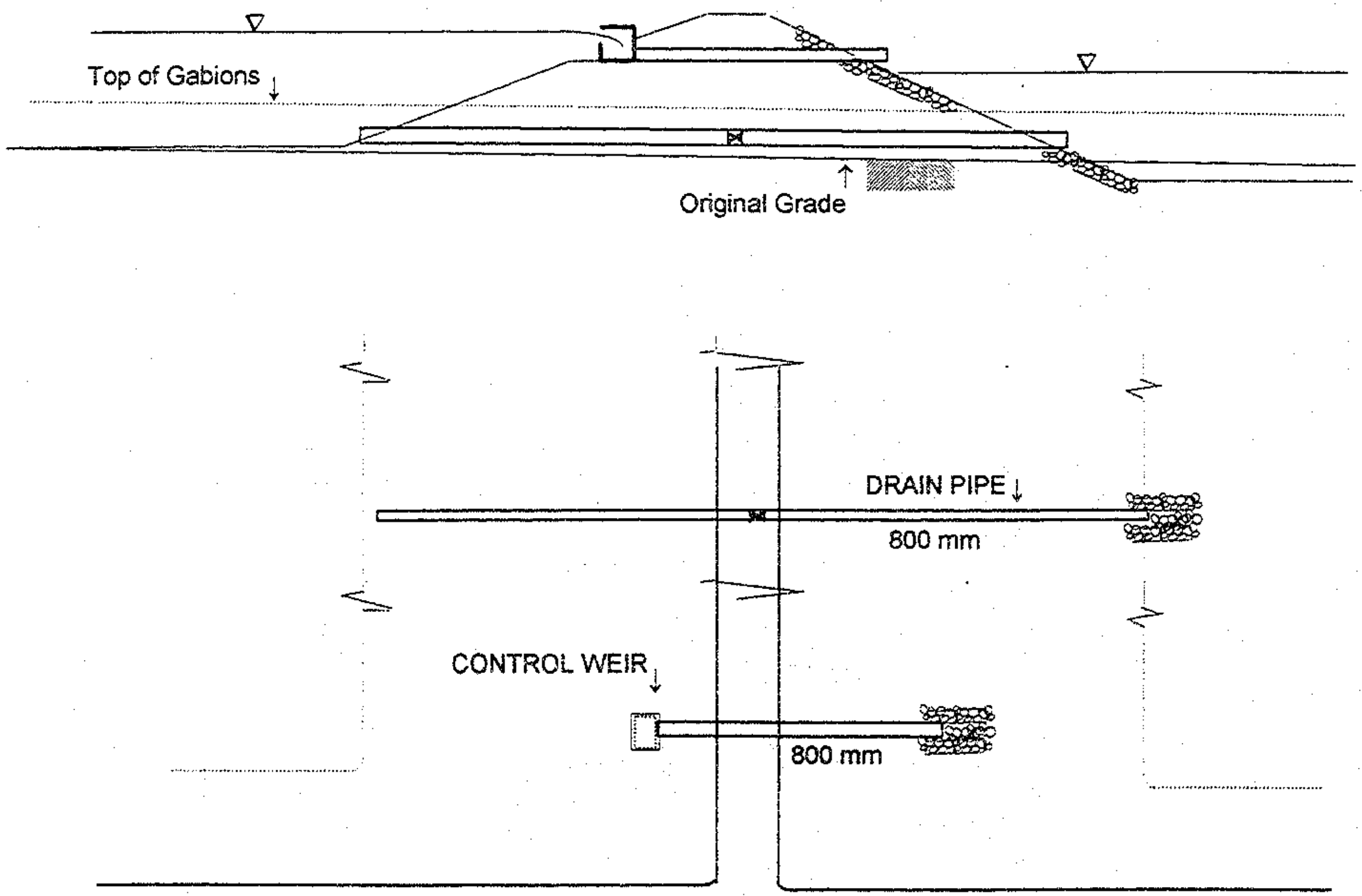

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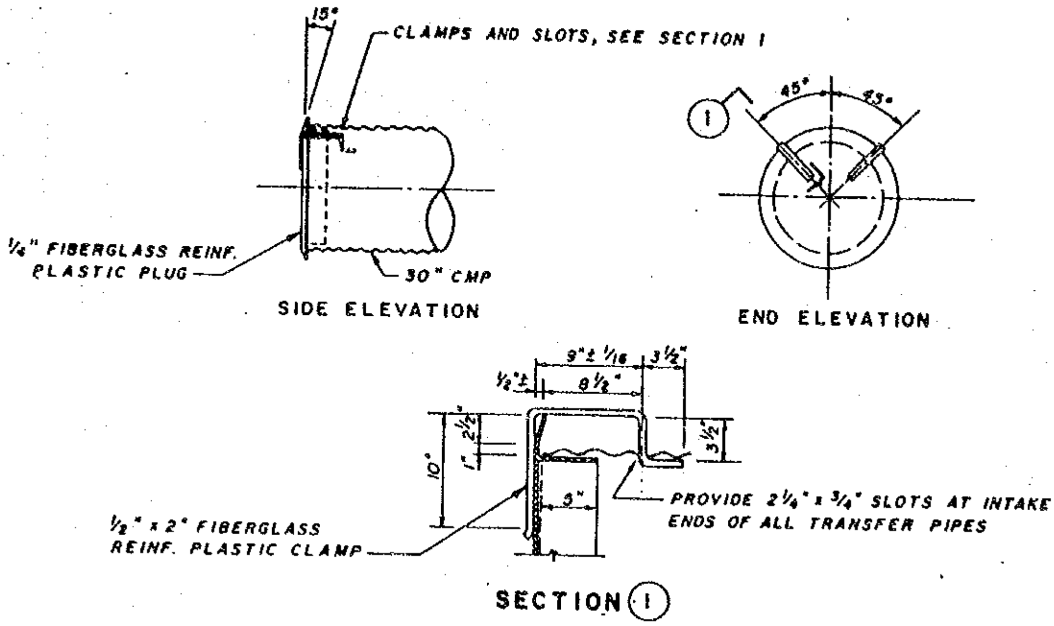
FIGURE D-3
SPECIAL FIBERGLASS PLUG



TYPICAL PLUG DETAIL
NO SCALE

FIGURE D-4
SITE 2 - PRELIMINARY LAYOUT OF SEDIMENTATION BASINS
SECOND STAGE CONSTRUCTION

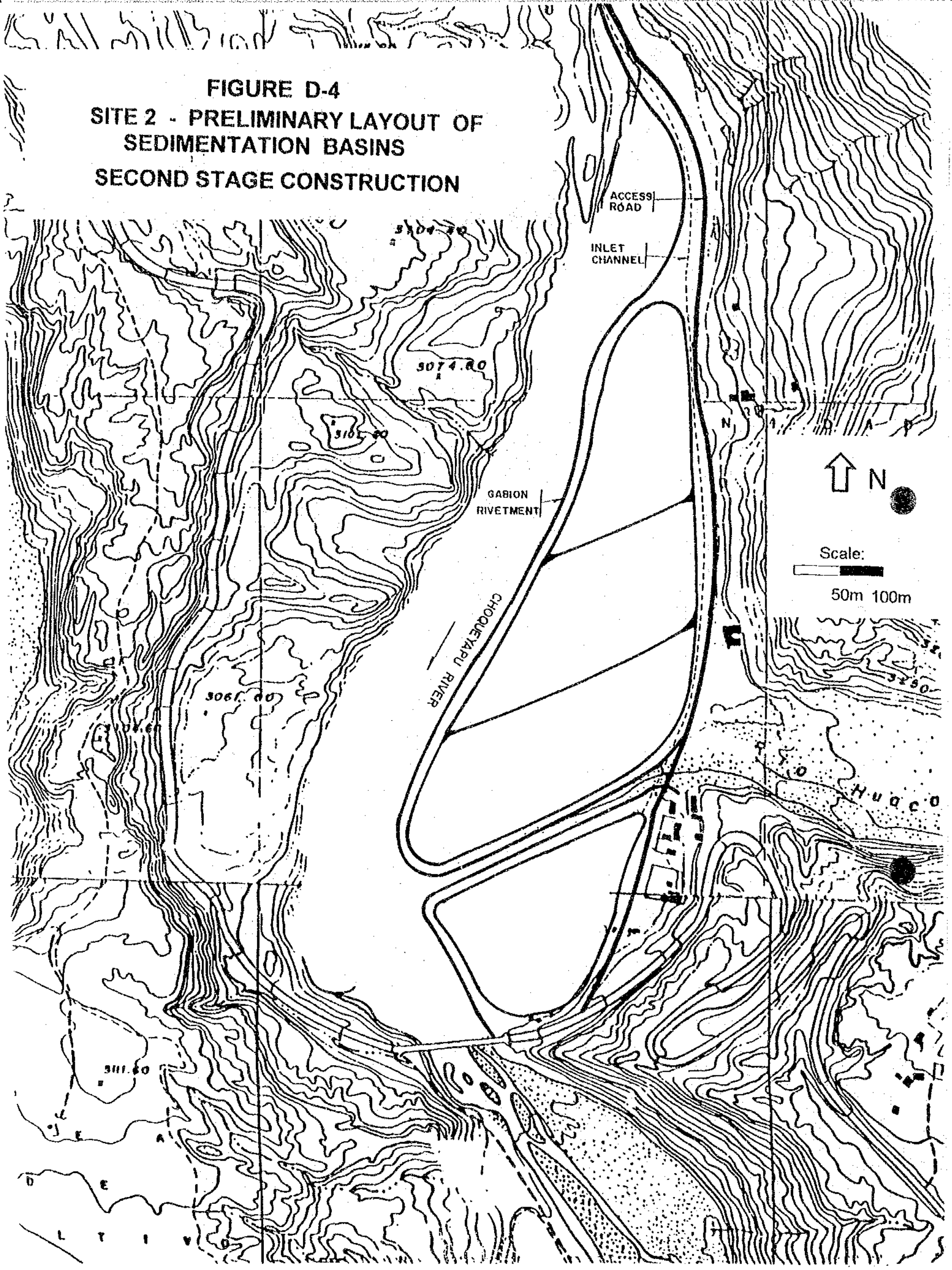


FIGURE D-5

PROFILE OF SEDIMENTATION BASINS

SITE 2 - STAGE 2 WORKS

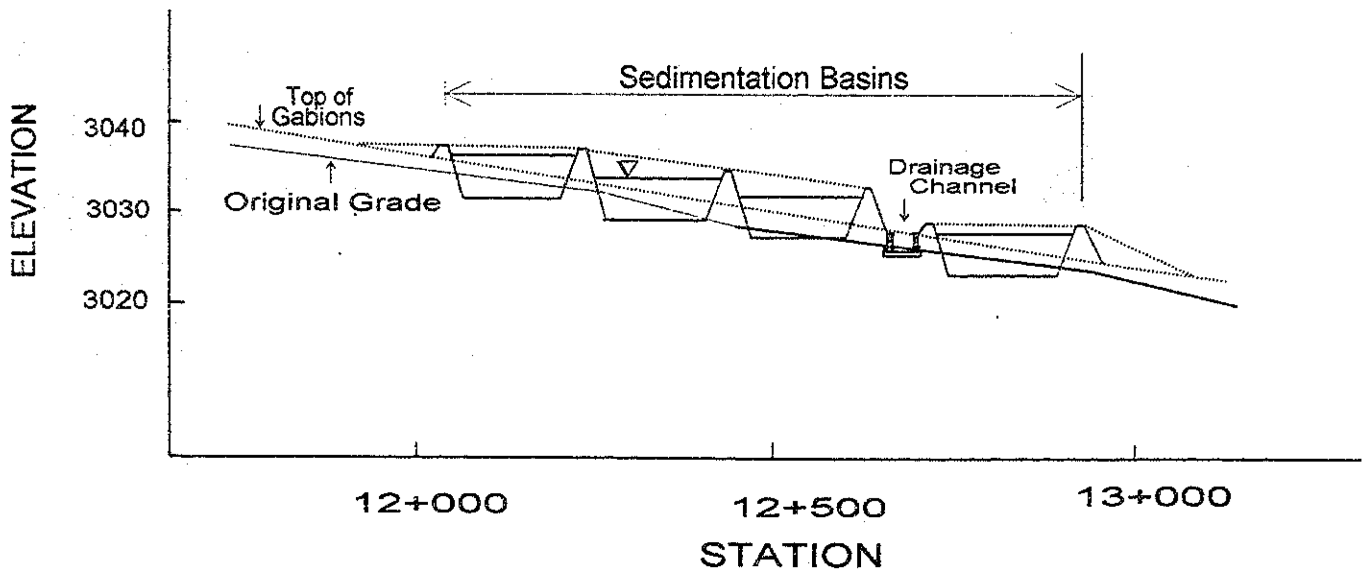


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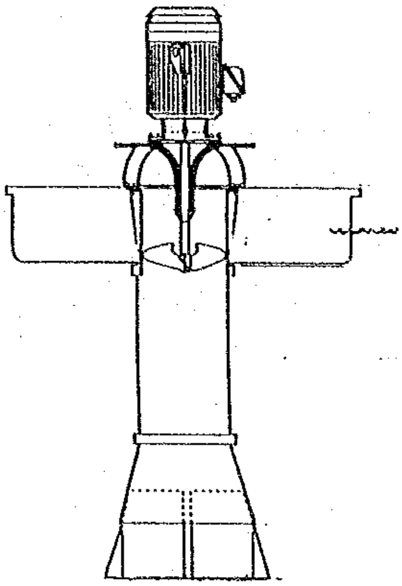
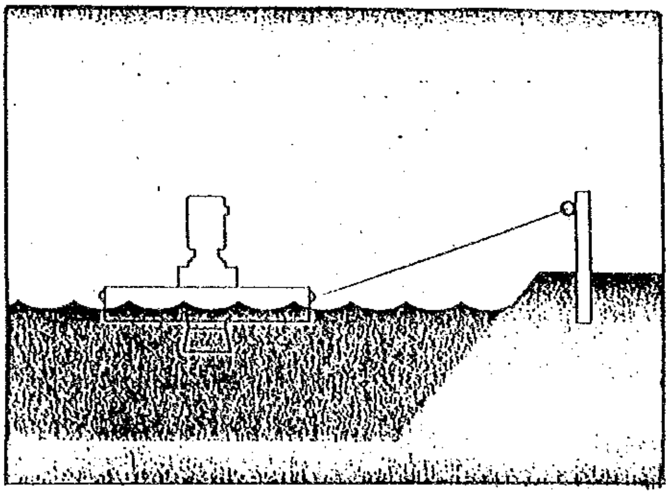
FIGURE D-6

PROPOSED AERATION EQUIPMENT



TYPICAL AERATOR WITH DRAFT TUBE

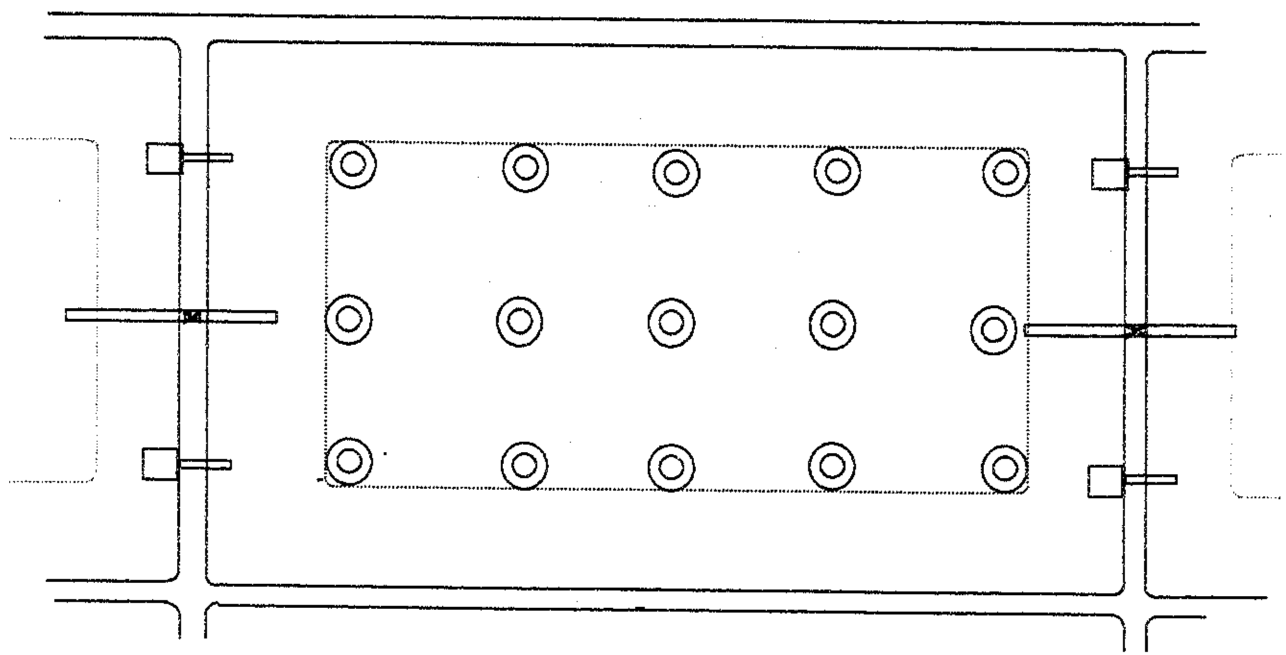


TYPICAL MOORING

FIGURE D-7

AERATION EQUIPMENT

TYPICAL LAYOUT - COMPLETELY MIXED LAGOONS



JICA